BEHAVIOR OF REINFORCED HIGHER-STRENGTH CONCRETE BEAMS IN BENDING AND SHEAR

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CHAPTER 1

INTROOUCTION

The term, high-strength concrete, is a relative one because the maximum strength specified has been changing over the past three decades. In the 1950's, concrete with a compressive strength of 5000 psi [34.4 MPa] was considered high-strength concrete. In the 1960's, concrete with a 6000 to 7000 psi [41.3 to 48.6 MPa] compressive strength was available commercially. In the early 1970's, a 9000 psi [62 MPa] mix was being produced, and in recent years, application of high-strength concrete has increased; and now it is used in many parts of the world. This growth has been possible as a result of recent developments in materials technology and the demand for high-strength concrete.

Concrete, based on its compressive strength, is classified here as follows:

Classification	Strength Ranoe
Normal-strength	2500-6000 psi (17.2 MPa-41.4MPa)
Higher-strength	6000-12000 psi (41.4 MPa-82.7 MPa)
High-strength	greater than 12000 psi (>82.7 MPa)

In this report concrete with a compressive strength of 12000 psi [82.7 MPa] is referred to as higher-strength concrete.

Present Usage of Higher-Strength Concrete

The main advantage in using higher-strength concrete is that it has a greater load-carrying capacity:

1. It has the potential to carry more load at a lower cost.

2. Reducing the dimensions leads to a smaller dead-weight of the structure, which means a more effective use of the available materials. This is particularly the case for pre-stressed members and compression members.

But, higher-strength concrete cannot altogether replace normal-strength concrete. For example, slabs which are made from a higher-strength mix will be very thin and will not be able to meet the ACI maximum allowable deflection specifications.[15]

Objectives

The objectives of this research project are:

1. to study the compressive and flexural behavior of higher-strength concrete (12000 psi [82.7 MPa]) made with aggregates available locally,

2. to determine the stress-strain relationship of higher-strength concrete;

3. to determine the shape of the compressive stress block of this concrete;

4. to determine the modulus of elasticity for this higher-strength concrete;

5. to determine Poisson's ratio for this higher-strength concrete.

CHAPTER 2

SELECTION OF MATERIALS

Introduction

Higher-strength concrete requires the highest quality materials. When making higher-strength concrete, one should also try to make use of the locally-available materials to ensure economy.

Cement

The choice of Portland cement for higher-strength concrete is extremely important. There are a few factors that are considered when choosing the right grade of cement, such as chemical composition (ASTM C-114), fineness(ASTM C-115), and cube strength (by the ASTM standard method of test C- 109).[7] In addition, Portland cement strength may vary from plant to plant; even in the same plant it, may vary from batch to batch. In all the operations involved, the compressive strength test results should be used as a check.

It is recommended that the final decision on the brand of cement be made based on the compressive strength of the trial mixes of the same workability at 28, 56, and 91 days.[4]

Coarse Aggregate

The selection of the coarse aggregate is the next most important item after choosing the cement. The behavior of coarse aggregate has a great influence on concrete strength.

Strength up to 5000 psi [34.5 MPa] depends essentially on the quality of the hardened cement paste holding the coarse aggregate together.[4] The aggregate has a much higher compressive strength than the cement paste.[4] It is important to consider the following factors when selecting a coarse aggregate for any concrete, and particularly so for higher-strength concrete:

a) strength,
b) maximum size and gradation,
c) particle shape and surface texture,
d) mineralogy and formation,
e) aggregate cement bond.

a. Strength

The aggregate chosen for higher-strength concrete should have a compressive strength at least equal to the hardened cement paste.[4] Since the crushing strength of many good quality aggregates available today generally exceeds 12000 psi [82.7 MPa], this factor is not a major problem for the production of higher-strength concrete.

b. Maximum Size and Gradation

Several researchers[2,3,11] have shown that in higher-strength concrete the compressive strength increases when the maximum size of the aggregate decreases. However, it is obvious that there should be some limitations in order to keep drying, shrinkage, and creep to a reasonable and practical value. A maximum aggregate size of 0.4 in. (10 mm.) is recommended in most cases.[11] Generally smaller size aggregates provide the most efficient use of the cement in the concrete.[4] This is due to the increase in the surface area which increases the bond strength. However, the optimum size of aggregate varies from mix to mix, and a trial batching should be used to find the optimum value.[4]

c. Particle Shape and Surface Texture

Carrasquillo[4] indicates that the ideal coarse aggregate for higher-strength concrete appears to be clean, cubical, angular, and 100 percent crushed stone with minimum flat sizes and elongated particles. Crushed stone aggregates produce a higher-strength concrete than rounded aggregates.

Coarse aggregates used in higher-strength concrete as in all concrete, should be free of dust coating. Any dust content causes an increase in fines and a consequent increase in the mixing water to achieve required workability.[4] This decreases the strength of the mix; therefore washing of the aggregates, if possible, is recommended.

d. Mineralogy and Formation

The compressive strength of concrete increases when a crushed stone aggregate is used.[4] This is not only due to the shape of the aggregate but also due to its mineralogy. Tests made by Parrot[18] on the effect of the type of coarse aggregate on concrete revealed the following: the effect of aggregate type upon strength was negligible at seven days; and the 28 and 90 day strengths did not appear to depend upon specific gravity, absorption, or acidity of the aggregate, but there seemed to be some dependence upon rock formation. Extrusive rocks generally have high-strength and a small grain size. It was noticed that as concrete becomes older, the incidence of aggregate fracture in broken pieces of concrete increases.[4] Therefore, the quality of an aggregate can be a significant factor governing the concrete strength.

e. Aggregate-Cement Bond

The aggregate-cement bond is the deciding factor of strength once the material hardens. The aggregate-paste bond decreases with increasing water-cement ratio and decreases with increasing maximum size of the aggregate.

In this project, quartzite stone with 3/4 in. [19 mm] maximum size, from Lincoln, Kansas was used, because this was available even though 10 mm is optimum.

Fine Aggregate

The gradation and the particle shape of fine aggregate are very important factors in production of a higher-strength concrete mix.

One of the important functions of fine aggregate in conventional concrete is its role in providing workability and good surface finish. Since the higher-strength mix has a higher cement content, the role fine aggregate plays in providing workability and good finish is not so crucial. Fine aggregates with a fineness modulus of 2.7 to 3.2 have been most satisfactory. In this investigation, Kaw River sand passing through sieve No.4 was used. This sand had a fineness modulus of 3.03.[15]

Water

Water that meets ASTM C-94[6] has no harmful effect on higher-strength concrete: therefore, this water is adequate. The ASTM standard C-94 gives the following specifications for the water to be used in mixing:

"The mixing water shall be clear and apparently clean. If it contains quantities which discolor it, or make it smell, taste unusual, be objectionable, or cause suspicion, it shall not be used unless a service record of concrete has been made from it or other information indicates that it is not injurious to the quality of concrete."

Admixtures

Due to an extremely low water-cement ratio, higher-strength concrete has an extremely low workability and slump. A chemical admixture called super-plasticizer, or super-water reducer can be used to improve the workability of concrete. This admixture actually reduces the angle of friction between water and the solids and causes the mix to be more workable. This effect is for a limited time only. The mix returns to its original slump after a short time. This action of the super-plasticizer makes it possible to have a mix with a high workability when fresh and a high compressive strength at the hardened stage. The amount of super-plasticizer required should be determined by trial mixes only. At any given water-cement ratio, the amount of super-plasticizer required to produce the required slump can be decided from the trial mix. In this project, 240 ml to 320 ml (8 oz to 11 oz) have been used per cubic foot of concrete.

CHAPTER 3

STUDY OF THE COMPRESSIVE STRESS BLOCK

Introduction

The equivalent rectangular stress block permitted by the ACI code 318-83[1] was based on beam tests with a compressive strength of 3000 psi [20.7 MPa.] to 6000 psi [41.4 MPa.][15]. The code recommends a reduction of .05 in the B_1 value for every 1000 psi [6.9 MPa] increase in compressive strength of concrete above 4000 psi [27.6 MPa] at which B_1=0.85. [The B_1 value is used in the calculation of the depth of the stress block.] This leads to a stress block with zero depth for 21000 psi [144.7 MPa] concrete which is an obvious fallacy. [In 1975, a lower limit of 0.65 for B_1 for concrete with strength higher than 8000 psi [55.1 MPa] was suggested by the code.[1]] Now, concrete mixes with a compressive strength of 8000 psi [55.1 MPa] and greater are used frequently in structures. Therefore, there is a strong need to evaluate the validity of the rectangular stress block assumption for higher-strength concrete.

Previous Work

The publications concerning higher-strength concrete are not only limited, but also contradict each other in their conclusions.

For example, in the work done by Rajagopalan, Leslie, and Everard, [10] it is concluded:

(i) The ACI building code rectangular stress block does not predict the behavior of beams with f'_c above 8000 psi [55.1 MPa].

(ii) Further research is warranted with respect to maximum strain in concrete when f'_c exceeds 8000 psi. [From Nedderman's[10] tests, the ultimate strain for concrete was found to be in the range of 0.00225 to 0.00285. The paper discusses these results, and it suggests that with increasing compressive strength, the maximum concrete strain becomes smaller.]

(iii) Pending further test results, a triangular stress block with extreme fiber stress at f'_c and zero atthe neutral axis is recommended as a conservative model for predicting the behavior of beams with f'_c above 8000 psi [55MPa].

a) From Nedderman's results,[10] showing the stress-strain relationships for concrete with a compressive strength of 12000 psi [83 MPa], it is observed that the stress strain curve is steeply ascending. It is almost linear up to the maximum strain, in marked contrast to the stress-strain curves of lower strength mixes which have a descending part past the maximum stress. This justifies the elastic theory and leads to the assumption of a triangular stress block.

In the work done by Nikaeen [15], he concludes:

i) "The shape of the stress block changes from rectangular to triangular as the strength increases. The centroid of the stress block lies at a distance of 0.37c from the top fiber (This value is very close to 0.33 which is the centroidal distance of the triangular stress block rather than 0.5 which is the centroidal distance of a rectangular stress block.)"

ii) "The strain behavior in high-strength concrete is different from normal-strength concrete, because strain at the ultimate condition is less than 0.003 in./in., and it decreases drastically with time. Therefore, a more conservative strain value of 0.002 in./in. is recommended."

But, in the work done by Wang, Shah, and Naaman,[21] they conclude:

1. "Rectangular stress distribution gives sufficiently accurate predictions of the ultimate loads and moments of reinforced concrete beams and columns made with higher-strength concrete."

2. "The value of the maximum concrete compressive strain at ultimate was always higher than 0.003 in./in."

Therefore, more tests with application of different methods should be done in order to check which theory is valid, or perhaps testing of the ACI code formula's validity for higher-strength concrete should be made. One of the main objectives of this work is to determine the shape of the compressive stress block.

Modulus of Elasticity

The ACI code suggests the modulus of elasticity of normal-weight concrete to be, [1]

$$E_c=33\ W^{3/2}\sqrt{f'_c}\ \text{psi.}$$

where f'_c is the 28-day cylinder compressive strength.

In tests done at Cornell University the E_c values obtained were found to be lower than those given by the ACI code formula[16].

The reason for this unconservative prediction may be the following. The strength of concrete is controlled mainly by the strength of the mortar.[16] The stiffness of concrete is influenced by both the mortar and the aggregate.[6] Therefore, an increase in the quality (strength and stiffness) of the mortar will significantly increase the strength of concrete without a directly proportional increase in the stiffness.

It is intended to determine the E_c value of higher-strength concrete (12000 psi) using cylindrical specimens. Two 3 in. X 6 in. cylinders were used in this test.

Poisson's Ratio

Poisson's ratio for higher-strength concrete was determined using 3 in. X 6 in. cylindrical specimens tested at 60 days. Strains were measured using four strain gages with their axes placed at 90 degrees to each other. The ratio between the transverse strain and the longitudinal strain for any given loading was calculated to give the poissons ratio.

Test Specimens and Method of Testing

Tests were conducted on rectangular beams to study the compressive stress distribution in a section at midspan. Each beam spanned 7 ft. [2134 mm] (the actual length of the beam was 7.5 ft [2286 mm],) and had a cross section of 8 in. X 12 in. [203 X 305 mm].

Three different types of specimens were made with varying steel ratios. One specimen was made in each type. These were:

a) two beams designed to fail in shear with p_{max}=0.5 p_{bal}. The shear reinforcement details were varied to study the shear capacity of the higher-strength concrete.

i.)One specimen was designed with no shear reinforcement. (SS1B)

ii.) One specimen was designed with partial shear reinforcement. (SS2B)

b.) an under-reinforced section with p_{max}=0.5p_{bal}. Shear reinforcements were provided in the beam to prevent possible shear failure. (UR1)

c) an over-reinforced section with p_{max}=1.5 p_{bal}. Shear reinforcements were provided. (OR1)

The detailed designs are given in Appendix I and the reinforcement details are given in Figs. 3.1 through 3.8.

Tension tests were done on the #3, #4, #7, #9 rebars which were used in the experiment. The results and the average tensile strength values are given in Table 3.1. These values are used in all moment calculations.

a) The beam was tested in third-point loading. It was supported on rollers at the ends to avoid any friction. The loading setup is shown in Fig. 3.9. The strains were measured with electrical strain gages chosen from the Micro Measurements Hand Book[12].

Electrical resistance strain gages, EA-06-7500T-120, were used on all the beams, and they were also used on the cylinders for longitudinal strain measurement. For transverse strain measurement in the cylinder, gages EA-06-250BB-120 were used. A high-speed data acquisition system was used to obtain and record the strain readings.

b) For determining the modulus of elasticity, cylindrical specimens of size 3 in. X 6 in. (76 mm X 152 mm) were tested according to ASTM standards[6]. Two gages were used to measure the strain in each cylinder. A total number of two specimens was used.

c) To measure Poisson's ratio, cylindrical specimens of 6 in. X 12 in. (152 mm X 305 mm) were used. Strains were measured using four strain gages with their axes placed at 90 degrees to each other.

d) The axial compressive strength was measured using two 6 in. X 12 in. (152 mm X 305 mm) cylinders and two 3 in. X 6 in. (76 mm X 152 mm) cylinders. Strains were measured using electrical resistance strain gages.

Strain Measurements

The strains in the test beams were measured with electrical resistance strain gages and mechanical strain gages. The gages were placed in the locations shown in Fig. 3.10, Appendix IV.

At the middle third of the beam, there was no strain gradient in a plane parallel to the neutral axis; therefore, 0.75 in. (18 mm)long gages can be used to measure strains. Two gages measured the extreme fiber strains and three gages measured the strain gradient along the beam's depth. A total number of eight gages was used per beam except for beam SS1B in which fourteen gages were used.

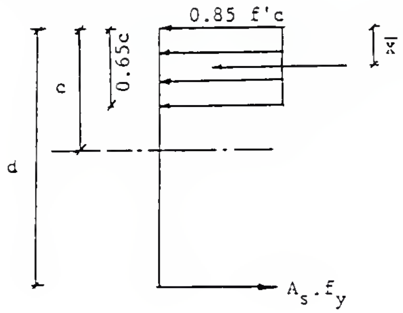
Prediction of the Moment Capacity

Under-Reinforced Beam

To predict the ultimate moment capacity of the under-reinforced beam, the steel was assumed to have yielded under the load. The total tensile force was calculated using the yield stress of the #7 bars and the area of steel in the beam. The total tensile force was equated to the total compressive force. The total compressive force was calculated using the rectangular stress block assumption, triangular stress block assumption, and parabolic stress block assumption.

a. Rectangular Stress Block

The equivalent rectangular stress block based on ACI 318-83 [1] is as shown:



Equating the total tensile force to the total compressive force,

$$A_s.f_y=0.85\ f'_c.(0.65\ c).b \qquad ...\ (3.1)$$

where A_s= area of steel,

f_y= yield strength of steel,

f'_c =compressive strength of concrete,

c=depth of neutral axis from top,

b= width of the beam.

Using this equation, the c value, i.e., the depth to the neutral axis, was determined from the top. The ultimate moment was calculated by the equation

$$M_u=0.85.f'_c.(0.65.c).b.(d-(0.65.c/2)) \qquad ...\ (3.2)$$

The calculated moment was compared to the actual moment taken by the beam.

b. Triangular Stress Block

The total tensile force was equated to the total compressive force calculated using the triangular stress block assumption. The average cylinder compressive strength and the depth of the neutral axis were used as the sides of the triangular stress block. Therefore

$$A_s.f_y=0.5.f'_c.c.b \qquad ...\ (3.3)$$

The depth of neutral axis was determined from this

The ultimate moment was calculated using the formula,

$$M_u=0.5.f'_c.c.b(d-(c/3) \qquad ...\ (3.4)$$

The calculated moment was compared to the actual moment.

c. Parabolic Stress Block

The shape of the stress block at the ultimate load given in Ref. 14 was used to predict the moment in the beams tested here using the assumption of a parabolic stress block. Stress values at various levels of all four beams were used to calculate the average stress values at those levels. Using these stress values, a parabolic curve was fitted to give the stress value at any given depth.

The total tensile force was equated to the total compressive force. The total compressive force was calculated by integrating the equation describing the stress block. The depth of the neutral axis was then calculated.

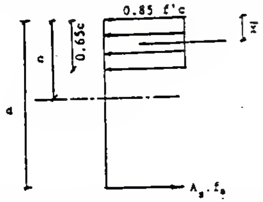
The centroid of the compressive stress block was also found by direct integration. The total moment was calculated using these quantities, and was compared to the actual moment taken by the beam. Detailed formulas and calculations are given in Appendix II

Over-Reinforced Beam

To predict the ultimate moment capacity of the over-reinforced beam, the total compressive force was equated to the total tensile force. The total compressive force was again calculated using the rectangular stress block assumption, triangular stress block assumption, and parabolic stress block assumption.

a. Rectangular Stress Block

As before, the ACI[1] method and assumptions were used.



Equating the total tensile force to the total compressive force gives

$A_s . f_s = 0.85\ f'_c . (0.65\ c) . b$

where A_s= area of steel,

f_s= stress in the steel,

f'_c =compressive strength of concrete,

c=depth of neutral axis from top,

b= width of the beam.

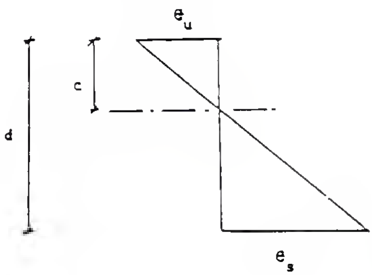
To find the tensile stress f_s, the strain compatibility equations are used. An ultimate concrete strain of 0.0025 in./in. was assumed.[14,15] From the strain compatibility condition shown,

$$\frac{\epsilon_s}{(d-c)} = \frac{\epsilon_u}{c} \qquad \ldots (3.5)$$

$$\epsilon_s = \frac{\epsilon_u (d-c)}{c}$$

$$f_s = E_s . \epsilon_s \qquad \ldots (3.6)$$

$$= \frac{E_s . \epsilon_u (d-c)}{c} \qquad \ldots (3.7)$$



E_s is taken as 29 X 10^6 psi.[1]

Equating the total tensile force and the total compressive force by substituting eq. (3.7) in eq. (3.1)

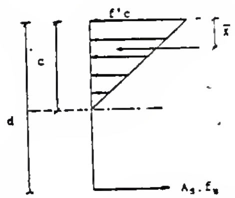
$$A_s.E_s.\epsilon_u((d-c)/c)=0.85.f_c'.(0.65c).b \quad . . . (3.8)$$

Solving this equation for c determines the depth of the neutral axis.

The moment taken by the beam was calculated from eq. 3.2 and compared to the actual moment taken by the beam. Detailed calculations are given in Appendix II.

b. Triangular Stress Block

The triangular stress block was assumed to have outer fiber stress equal to the compressive strength f_c' and the stress distribution shown.



Equating the total tensile force to the total compressive force gives

$$A_s.f_s=0.5\ f_c'.c.b$$

where A_s= area of steel

f_s= tensile strength of steel

f_c'=compressive strength of concrete

c=depth of neutral axis from top

b= width of the beam.

To find the tensile stress f_s, the strain compatibility conditions are used as before. An ultimate concrete strain of 0.0025 in./in. was assumed[14,15], and ϵ_s is given by Eq 3.5 and f_s is given by Eq. 3.7. As before, E_s is taken as 29 X 10^6 psi. Equating the total tensile force and the total compressive force gives,

$$A_s . E_s . \epsilon_u((d-c)/c) = 0.5 . f'_c . c . b \qquad . . . (3.9)$$

from which c is determined.

The moment taken by the beam was calculated from Eq. 3.4 and compared to the actual moment taken by the beam. Detailed calculations are given in Appendix II.

c. Parabolic Stress Block

The procedure described for the under-reinforced beam was used except the steel stress, f_s is given by Eq. 3.7.

The centroid of the compressive stress block was found by direct integration. The total moment was calculated using these quantities and was compared to the actual moment taken by the beam. Detailed calculations are given in Appendix II.

CHAPTER 4

PROPORTIONING, MIXING, AND PLACING

Introduction

The objective in designing a concrete mix is to obtain a material which will possess certain desired properties such as workablity, finishability, etc. when plastic and required characteristics such as strength, durability, wear resistance, and water tightness when hardended, at the lowest possible cost. Proportioning of the higher-strength mix requires more accuracy than is usually needed for normal strength mixes, because optimum performance is required from each component.[4]

Mix Proportioning

Since higher-strength concrete technology is relatively new in the field, there are no conventional mix design methods available for strengths higher than 8000 psi [55.1 MPa]. A trial batch program is the most effective method for determining the suitability of the materials and their proportions for a specifc job.[4]

The factors that decide the strength of the mix are:

1) water cement ratio
2) cement content
3) aggregate content
4) admixtures.

Water-Cement Ratio

To obtain a mix with high compressive strength, using a given set of materials, the lowest possible water-cement ratio should be used together with a minimum amount of mixing water.

According to Abram's law,[17] for any given condition of the test, the strength of a workable* concrete mix is dependent only on the water-cement ratio. The lower limit for the amount of water will be that amount necessary to allow the hydration of Portland cement to go to completion. Portland cement requires about one-fifth to one-fourth of its weight of water to become completely hydrated. Then if the water-cement ratio is below 0.4, complete hydration cannot be secured.[17]

It has been found, nevertheless, that strength increases with a reduction in water-cement ratio to a value of 0.2 or even lower and it appears only the outer surface of each cement particle will become hydrated.[17] Any reduction in water cement ratio up to 0.2 will increase the strength of the mix considerably. For this study, a water-cement ratio of 0.28 was used. Trial mixes were done in the region of 0.26 to 0.3 in which there is a considerable increase in strength for a small reduction in water-cement ratio.[15]

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* The term workable refers to the ability of the mix to be compacted to such an extent that the air content is less than 2 percent, which is easily made possible with the use of super-plasticizers even in the very low water-cement ratio mixes.

Cement Content

The mix proportions should be determined for production of a concrete mix with the lowest water requirments and the highest required compressive strength for the specified workability. This leads to mixes with high cement factors. However, there is an optimum cement content above which addition of any amount of cement will not appreciably increase the strength. This amount depends on the aggregate type, aggregate size, mixing conditions, slump level, and the amount of air entrained. Freedman[7] states that a trial mix for 10000 psi [68.9 MPa] can contain about 940 lb per cubic yard (557 kg per cubic meter). Any further increase in cement content above this level will not result in an appreciable increase in compressive strength. Such high cement factors are inevitable in proportioning for the high strength concrete. The mix used in this work has a cement content of 864 lb per cubic yard [512 kg per cubic meter].

Aggregate Content

To obtain a mix of reasonable workability and to retain a low water-cement ratio, the cement paste must not be combined with excess aggregate. Here, the mix used in a previous work[15] was found to have the optimum amount of aggregates to produce the required workability and the ability to give the required strength for the mix. The amount of aggregates used are given in the mix design in Appendix II.

Mixing and Placing

Since the water used in the mix is just enough to hydrate the cement, the mixing water has to be used effectively. Initially, it was decided to mix the cement and water for a minute in the mixer to make a slurry, and then aggregates were added to the slurry. This method of mixing ensures that the cement receives all the water it requires for hydration.

One batch of six 3 in. X 6 in. cylinders was made using this mixing technique. The average compressive strength of these cylinders was compared to the average compressive strength of the concrete made by the regular mixing process. The study showed an increase in the compressive strength of the mix. The results of the cylinder compressive strengths are given in Tables 4.1 and 4.2. It is not certain to what extent the strength is increased (the test results showed an apparent increase of 1.5 percent in the average compressive strength, but later the loads shown by the dial indicator were found to be wrong due to the failure of a high-pressure valve in the testing machine).

Later it was decided to follow the regular mixing procedure. All the solids were mixed, including cement, for four minutes. After the complete mixing was ensured, water was added slowly over a period of two minutes, followed by vigorous mixing for two to four minutes.

Since the capacity of the mixer was three cubic feet, two batches were mixed for each beam. Good compaction was ensured by using a rod vibrator. Cylinder samples of 3 in. X 6 in. were made for both batches of the mix. The same operators were used in each batch of all four beams to minimize the amount of human error involved. The top surface of the beam was given a smooth finish by working it with a glass plate .

Curing

The formwork was removed after 24 hours, and the beam was cured until seven days prior to testing. The curing was done by pouring water on the beam at regular intervals and keeping it covered by a plastic drop cloth to reduce the evaporational loss. All four beams were tested at an age of 60 days from the day of casting.

CHAPTER 5

BEAM TEST AND RESULTS

Introduction

All four beams were tested in a universal testing machine (Tinius Olson) with maximum loading capacity of 200,000 lb [890 kN], which can be read accurately to the nearest 20 lb [89 N]. A nearly uniform loading rate was applied for all the beams.

Test Setup

The beam was supported on two rocking, roller edges. To avoid the line contact which might cause higher bearing stress and a bearing failure, two plates of 3 in. X 12 in. X 1 in. [76.2 X 304.8 X 25.4 mm] were used on the roller edges. This ensured a normal reaction from the supports at all loads. The load from the machine head was transferred to the beam at the third-points by a steel loading beam, shown in Fig. 3.9. The loading points were seated on a hydro-stone mortar coating to ensure uniform load transfer. Supports, bearing plates, the test beam, and the loading beam were checked for any possible eccentricity in two directions to avoid any possible torsion introduced into the beam.

Strain readings were obtained by eight electrical-resistance strain gages placed on the beam (in the first beam, a total number of 14 gages were used, but later it was decided to use only eight gages for each beam). The gage locations and the numbering sequence are shown in Figs. 3.5 and 3.10. The strain gages were connected to a high-speed data acquisition system. Deflections were measured by a dial gage with a least reading of 0.001 in.(0.025 mm.) at the mid point of the beam.

To measure the strain in the tensile zone, a Whittemore gage with a gage length of 8 in. was used. The gage end points were brass buttons with a concentric hole of 1/16 in. that were made in the lab. Buttons were glued to the beam by epoxy at 8 in. and 10 in. from the top.(see Fig. 3.5)

The 3 in. X 6 in. cylinder samples for each beam were tested on the same day as the beam. The first trial mix (taken from Reference 15) compressive strength results are given in Table 5.1. The cylinder compressive strength results (modified mix) for each beam are given in Tables 5.2, 5.3, 5.4 and 5.5. The proportions of the modified mix are given in Appendix II.

Testing Procedure

Suitable loading increments were used depending upon the estimated ultimate capacity of the beam. After each increment, the readings from the strain gages and the dialgage were taken. The beam was checked for visible cracks and the cracks were marked up to the leading edge.

Results and Discussion

The strain readings obtained from the electrical-resistance gages, Whittmore gages and the corresponding load deflection values are tabulated in Tables 5.6 through 5.16. The strain values for corresponding loads in the compression zone are plotted across the depth of the beam in Figs. 5.1, 5.2, 5.3 and 5.4, respectively, for Shear Specimen I(SS1B), Shear Specimen II (SS2B), Under-reinforced Specimen (UR1) and Over-reinforced Specimen (OR1). The depth of the neutral axis for each beam was calculated by assuming a uniform strain distribution across the depth of the section and is shown on each plot.

The 3 in. X 6 in. (76 mm X 152 mm)concrete cylinder samples were tested at the same age as the beam; the cylinder stress-strain data are given in Tables 5.17 and 5.18. Using the cylinder stress-strain data, a third degree polynomial was fitted to determine a stress strain relation. Using this polynomial, each stress corresponding to the beam's measured strain values was calculated. These stress values are plotted through the depth of the beam,thus giving the shape of the stress block shown in Figs. 5.5, 5.6, 5.7 and 5.8.

For every loading, the total compressive force was estimated using the triangular stress block assumption. Using the extreme fiber stress as one side of the triangle and the depth of the neutral axis as the other, the area and the centroid of the triangular stress block were calculated. Neglecting the tensile strength of concrete, the lever arm up to the center of the steel reinforcements was calculated. The internal moment produced by this couple was compared with the actual moment taken by the beam at the corresponding load.

Similarly, the ACI equivalent stress block was used to calculate the internal moment at the ultimate load. For this stress block, 0.85 times the average cylinder compressive strength and 0.65 times the depth of neutral axis (B_1=0.65) were used. The internal moment was calculated and checked with the actual moment taken by the beam at ultimate load. Finally, the actual stress values through the depth to the neutral axis were used to fit a parabolic curve for the stress block. By integrating, the area of the stress block and the centroidal distance were calculated. Using these values, the internal resisting moment was calculated and checked with the actual moment taken by the beam.

The deflections up to the flexural cracking moment for each beam were calculated using the full uncracked moment of inertia of the section. A modulus of rupture of $6.5\sqrt{f'_c}$ was used to estimate the cracking stress. Beyond this stress, the beam was treated as a cracked section and the effective moment of inertia was used to calculate the deflections. This was calculated using the formula

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \cdot I_g + 1-\left(\frac{M_{cr}}{M_a}\right)^3 \cdot I_{cr} \leq I_g \qquad \ldots (5.1)$$

where M_{cr} = cracking moment,

M_a = maximum moment,

I_g = gross moment of inertia of the section,

I_{cr} = moment of inertia of the transformed section,

I_e = equivalent moment of inertia.

Using this I_e, the deflections of the beam were calculated and compared to the actual deflections of the beam under load. A small program in BASIC language was written to perform all the above mentioned calculations.

<u>Shear Specimen I</u>

The load and the corresponding stresses at the top, at two, four, and six inches from the top are given in Table 5.19. These values are plotted in Fig. 5.5. From this, it can be observed that the stress block has a negative curvature. This may be because the failure is due to shear and not flexure or it may be due to the errors in the measurement.

The actual moments, the calculated triangular stress block moments (using the actual extreme fiber stress), and the calculated parabolic stress block moments are compared in Table 5.20. From this, it can be observed that the moments calculated using the triangular stress block are found to be higher than the actual moments for higher loads. The parabolic stress block is able to predict the moment closely and conservatively with an error of 11 percent at failure.

The actual deflections and the calculated deflections are compared in Table 5.21.

<u>Shear Specimen II</u>

The load and the corresponding stresses at the top at two, four, six inches from the top are given in Table 5.22. These values are plotted in Fig. 5.6. From this it can be observed that this stress block also has a negative curvature.

The actual moments, the calculated triangular stress block moments (using the actual extreme fiber stress), and the calculated parabolic stress block moments are compared in Table 5.23. The parabolic stress block is able to predict the moment closely and conservatively with an error of 16 percent at failure. The actual deflections and the calculated deflections are compared in Table 5.24.

Under Reinforced Beam

The load and the corresponding stresses at the top, at two, four, and six inches from the top are given in Table 5.25. These values are plotted in Fig. 5.7.

The actual moments, the calculated triangular stress block moments (using the actual extreme fiber stress, and the depth of the neutral axis), and the calculated parabolic stress block moments are compared in Table 5.26. The triangular stress block estimate was in error by 9 percent at the maximum load (load 18).

The actual moment, the calculated moment using the rectangular stress block (using 0.85 times the average compressive strength of the 3 in. X 6 in. cylinder samples) and the calculated parabolic stress block moments are compared in Table 5.27 for the ultimate load. It is found that the rectangular stress block assumption gives a close and conservative estimate with 3 percent error. The parabolic stress block assumption estimates the actual moment with 42 percent error. The actual moment, the calculated rectangular stress block moment, and the triangular stress block moment are given in Table 5.28. The actual deflections and the calculated deflections are compared in Table 5.29. From the initial loading to the final failure, the neutral axis moved through a distance of nearly 2.58 inches.

Over-Reinforced Beam

The load and the corresponding stresses at the top, at two, four, and six inches from the top, are given in Table 5.30. These values are plotted in Fig. 5.8.

The actual moments, the calculated triangular stress block moments, (using the actual extreme fiber stress and the depth up to the neutral axis) and the calculated parabolic stress block moments are compared in Table 5.31. From this, it can be observed that the moments calculated using the triangular stress block are found to be much lower than the actual moments. The triangular stress block underestimates the ultimate moment of the section by 39 percent at failure. The parabolic stress block is able to predict the moment closely and conservatively with an error of 7 percent at failure.

The actual moment, the calculated moment using the rectangular stress block (using 0.85 times the average compressive strength of the 3 in. X 6 in. cylinder samples), and the calculated parabolic stress block moment at failure are compared in Table 5.32.

It is found that the rectangular stress block gives an estimate with 29 percent error. The parabolic stress block is able to give the actual moment with 7 percent error. The actual moment, the calculated rectangular stress block moment, and the triangular stress block moment at failure are given in Table 5.33. The actual deflections and the calculated deflections are compared in Table 5.34. From the initial loading to the failure, the neutral axis has moved through a distance of nearly 1.26 inch. This is due to the fact that tensile steel that has not yielded, and the very small cracks that are formed do not severely affect the location of the neutral axis.

Shear Behaviour

There have been a number of discussions on the correct relation between compressive strength and shear capacity. The current ACI code assumes that the nominal shear capacity is proportional to $(f_c')^{0.5}$. The work done by some investigators[13] conclude that for high-strength concrete the shear strength is proportional to $(f_c')^{0.333}$.

Two reinforced concrete specimens were made, one without any shear reinforcement, and the other with 62 percent of the required shear reinforcement based on the ACI (318-83)[1] method. The longitudinal reinforcement steel ratio was 0.5 times the balanced steel ratio. Details of reinforcing are given in Figs. 3.1, 3.2, 3.5 and 3.6. Crack patterns are shown in Figs. 5.9 and 5.10. The total span of the beam was 7 ft. and the shear span ratio was 2.5.

Each beam was tested in the same way as the flexural specimens. The strain data obtained and the corresponding load and deflections are given in Tables 5.6, 5.7, 5.8, 5.9, 5.10 and 5.11. During the testing of the second beam, SS2B, the spreader beam failed due to buckling at 60000 lb. The beam was reloaded the next day using a new spreader beam. In the beam without shear stirrups, SS1B, the diagonal cracking load and the ultimate load were used to calculate the critical shear force, and the ultimate shear stress using the effective depth of the beam, and the width of the beam. The shear force calculated was compared with ACI equation 11-3 and 11-6 [1] and Zsutty's[13] equation.

In the second beam, the amount of shear taken by the steel stirrups was calculated by considering the number of stirrups encountered by a major diagonal crack and then assuming the stirrups have yielded. The remaining shear was assumed to be taken by the concrete.

ACI Eq.(11-3)[1]

$$V_{cr}=2\sqrt{f_c'}\ b_w d \qquad \ldots (5.2)$$

ACI Eq.(11-6)[1]

$$V_{cr}=\left(1.9\sqrt{f_c'} + 2500p\frac{V_u d}{M_u}\right)b_w d \qquad \ldots (5.3)$$

Zsutty's Eq.[13]

$$V_{cr}=59\left(f_c' p\frac{d}{a}\right)^{0.333} \cdot b_w d \qquad \ldots (5.4)$$

At ultimate

Zsutty's Eq.[13]

$$V_{cr}=63.4\left(f'_c p\frac{d}{a}\right)^{0.333} \cdot b_w d \qquad \ldots(5.5)$$

The values calculated using these formulas are compared in Table 5.35. The actual ultimate shear force of the beam with no web reinforcement was found to be 21000 lb [93.5 kN]. The ACI formula gave a close and conservative prediction of the ultimate shear force as 17600 lb[78.3 kN]. Zsutty's equation over-estimated the ultimate shear capacity of the section to be 23000 lb [102.35 kN].

In the second specimen, with 62 percent web reinforcement, the amount of shear taken by the concrete was found to be 19650 lb [87.4 kN], which is predicted by the ACI formula closely and conservatively as 17600 lb [78.3 kN]. However, Zsutty's equation predicted 24400 lb [108 kN].

Ultimate Strains

From the test data in Tables 5.13, and 5.16, the average strains at the ultimate load condition for the under-reinforced and over-reinforced beams are 0.0024 in./in. and 0.0026 in./in. From Table 5.18, the maximum strain in concrete cylinder No.2 is 0.0025 in./in. These values show that the ultimate strain in high-strength concrete is less than 0.003 in./in. Ref. 5 and 16 have indicated similar results. An ultimate strain value of 0.0025 in./in. was recommended by Carrasquillo[5] et al.

Modulus of Elasticity

The uniaxial compressive stress-strain values given in Tables 5.17 and 5.18 were plotted (Figs. 5.13 and 5.14), and the point of 0.45 times the maximum cylinder compressive strength was determined. A straight line was drawn from the origin to 0.45 f'_c, and the slope of that line was used to calculate the secant modulus of concrete. The average modulus of elasticity was found to be 6.42 X 10^6 psi for this higher-strength concrete. The ACI formula,[1]

$$E_c = 33\, W^{1.5}\sqrt{f'_c} \qquad \ldots (5.6)$$

where W=dry unit weight of concrete, was used to predict the modulus of elasticity. The dry unit weight was determined by weighing six 3 in. X 6 in. cylinders, and the average dry unit weight was 153 pcf (2470 kg/m^3). The predicted value was found to be 6.78X10^6 psi. Thus the ACI formula overestimates the elastic modulus by nearly 6 percent. Detailed calculations are shown in Appendix II.

Poisson's Ratio

The Poisson's ratio was calculated from the stress-strain data of Cylinder No. 2 given in Table 5.18. The Poisson's ratio was found to be constant nearly up to failure. The value is approximately 0.12. The full set of values is given in Table 5.18. Poisson's ratio values are plotted in Fig. 5.15.

CHAPTER 6

SUMMARY OF RESULTS AND CONCLUSIONS

Summary

A mix design for high-strength concrete was developed using the available data from the mix design work[15] done earlier. The proportions of the mix are given in Appendix II. A superplasticizer was used in all mixes to improve the workability. A total number of four beam specimens was made and tested to determine the shear strength, and the shape of the compressive stress block. The test results and the analysis lead to the following conclusions.

Conclusions

1. Higher-strength concrete has a brittle mode of failure. In the cylinder specimens, no cracks were observed before failure. The failure was sudden and explosive. The failure cracks were vertical, from end to end of the cylinder. The failure plane was very smooth and did not discriminate between the aggregate and the matrix.

2. The compressive stress-strain curve is nearly linear up to failure.

3. Due to the various assumptions used, the ACI equivalent rectangular stress block is able to give a closer agreement with the actual measured ultimate moment for beam UR1 than the triangular stress block. But using the ACI rectangular stress block for concrete with a compressive strength of 8000 psi [55 MPa] or greater is not recommended because the shape of the

actual stress block appears to be triangular or parabolic at ultimate loads.

4. For beam OR1, the parabolic stress block assumption gives better agreement with the measured ultimate moment than the ACI rectangular stress block assumption or triangular stress block assumption.

5. The ultimate strain values for this higher-strength concrete are different from normal-strength concrete. The strain in direct compression and flexure are found here to be lower than 0.003 in./in. used in the ACI code. Therefore, a more conservative strain value of 0.0025 in./in. is recommended.

6. Nominal shear values predicted by the ACI code formulas were found to be close to the calculated, experimental values and conservative. Zsutty's equations overestimated the nominal and ultimate shear capacity of this concrete.

7. The average value of the modulus of elasticity of this higher-strength concrete (based on two samples) was found to be less than the value predicted by the ACI code.[1] More work has to be done to find the exact relation between the unit weight of concrete, compressive strength, and modulus of elasticity.

8. The deflections calculated using the ACI equivalent moment of inertia were found to be lower than the actual deflections. The deflections for all beams are plotted in Fig. 5.16 in which it is seen that UR1 has a more ductile behaviour than the other beams.

9. Poisson's ratio for higher-strength concrete was found to be about 0.12

APPENDIX I

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REFERENCES

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APPENDIX II

DETAILS OF SOME CALCULATIONS

Mix Proportions

Mix proportions obtained from Ref. 15:

All weights are lb. per one cubic foot.

Materials	Quantity
Cement	31.93 lb
Quartzite	49.65 lb
Sand	60.88 lb
Water	9.24 lb
Super-plasticizer	0.40 lb
Air % (est.)	0.5
Initial Slump, in.	2.5

Specific gravity of Sand = 2.63 (Ref. 15)

Specific gravity of Quartzite = 2.54 (Ref. 15)

Specific gravity of super-plasticer = 1.2 (Ref. 15)

The weight of the super-plasticizer is added to the weight of the water since the super-plasticizer is considered to act as water in the mix.

So the water-cement ratio of the mix is,

$$=(9.24+0.4)/31.93$$

$$=0.30$$

Test age, days = 47 days

Nominal compressive strength f_c' = 9400 psi

This mix had an inadequate compressive strength and it was decided to design a mix with lower water-cement ratio. The new mix proportions are given below.

All weights are lb. per one cubic foot

Materials	Quantity
Cement	31.93 lb
Quartizite	49.65 lb
Sand	60.88 lb
Water	8.28 lb
Super-plasticizer	0.74 lb
Air % (est.)	1.0
Initial Slump, in.	5.5

Specific gravity of Sand = 2.63 (Ref. 15)

Specific gravity of Quartzite = 2.54 (Ref. 15)

Specific gravity of super-plasticer = 1.2 (Ref. 15)

The water-cement ratio used in the project, including the weight of the super-plasticizer is calculated below;

$$=(8.28+0.74)/31.93$$

$$=0.28$$

Water-cement ratio (f'_c=12000[83 MPa])=0.28.

Test age, days = 60 days

Nominal compressive strength f'_c = 12000 psi

Oesign of Steel Reinforcement

Two specimens are designed to fail in shear in order to study the shear strength characteristics of higher-strength concrete.

Oesign of Shear Specimen I

$p_{MAX}=0.5\ P_{bal.}=0.02279$

$A_s=P_{max}.b.d.$

$A_s=(0.022792)(8)(11)=2.005525$ sq. in.

Use three, #7 bars in a row

Area provided=1.8 sq. in.

$M_u=A_s f'_y(d-C/3)d$ C=3.0 in. C/3=1.0 in.

$M_u=(1.8)(60)(11-1.0)$

$=1090.8$ kip-in.

$=\ 90.9$ kip-ft.

$M_u=Pl/6=90.9$, P=77.90 kip (span=7')

Shear max.=77.90/2=38.95 kips

Allowable shear taken by concrete=$\phi 2\ f'_c\ bd$

$=16.8$ kips

No shear reinforcement is provided for the beam.

Since the maximum shear is much greater than allowable shear, the beam is going to fail in shear.

Oesign of Shear Specimen II

In this beam, the same amount of steel is used as in the previous specimen.($P_{max}=0.5P_{balance}$, $A_s=1.8$ sq. in.)

From the previous design,

$M_u=86.4$ kip-ft., Ultimate load=74.1 kips

$V_u=37.1$ kips.

Shear taken by concrete $=\phi V_c=16.8$ kips.

Shear to be taken by steel $=(V_u-\phi V_c)=(37.1-16.8)$

$=20.3$ kips.

Using #3 bars for stirrups $A_s=0.11X2=0.22$ sq. in.

$$S=\frac{(1)(0.22)(60)(10.5)}{20.3}=6.83 \text{ in.}$$

An equal spacing of 11 in. c/c. was used in the second beam.

Shear reinforcement provided $=(6.83/11)100$

$=62.1$

The second speciman is designed with a shear reinforcement of about 62 percent with #3 stirrups placed at 11 in. c/c.

Design of Under-Reinforced Section

The section is under-reinforced using $p=0.5\ p_{bal.}$ for steel as the main reinforcement in the beam.

$p_{bal.}$ using triangular stress block

Using 60-grade steel,

$$C_b=d\left(\frac{\epsilon_u}{\epsilon_u+\epsilon_y}\right).$$

ϵ_y=yield strain in steel.

$E_s=29X10^6$ psi.

Multiplying the numerator and the denominator by E_s,

$$C_b=d\cdot\left(\frac{72.5}{72.5+60}\right)$$

$c_b=.5471$ d. (when $C=C_b$)

$A_s.f_y=.5\cdot f_c'\cdot(0.5471)\ b$ $\quad$ b=8in., d=10.5in.,

f'_c=10000 psi.[68.9 MPa]

f_y=60000 psi.[413.4 MPa]

$$\frac{A_s}{bd} = 0.0455975 = 0.0456$$

using 0.5 $p_{bal.}$, p_{max}=0.0228

<u>area of steel</u>.

$A_s = p_{max} \cdot b \cdot d$

$=(0.0228)(8)(10.5)$

$=1.92$ sq. in.

Use 3, #7 bars in a row

Area provided =1.8 sq. in.

<u>load calculation using triangular stress block</u>

For third-point loading:

Bending moment maximum =Pl/6

Maximum Shear =P/2

$p_{max}=0.5\ p_{bal}$

A_s=1.8 sq. in.

(Since the failure is going to be in the middle third of the beam, the steel at the top of the beam that is used to hold the stirrups in position is not present; it will not affect the design.)

d=10.5 in.

$$c = a = \frac{A_s \cdot f_y}{0.5 \cdot f'_c \cdot b} = \frac{(1.8)(60)}{(0.5)(10)(8)} = 2.7 \text{ in.}$$

$M_u = A_s . f_y (d - c/3) d$

$c = 2.7$ in, $c/3 = 0.9$ in.

$M_u = (1.8)(60)(10.5 - 0.9)$

$M_u = 1036.8$ kip-in. $= 86.4$ kip-ft

Max moment $= Pl/6$, $l = 7.0$ ft

Solving for P, P $= 74.1$ kips

Max shear $= P/2$ $= 37.1$ kips

Total shear to be taken by the beam = 37.1 kips

Shear taken by concrete $= \phi 2 \sqrt{f'_c}\, bd$

$= (1.0)(2)\sqrt{(10000)}(8)(10.5)/1000 = 16.8$ kips

<u>stirrup design</u>

Stirrups have to be designed for 37.1-16.8 = 20.3 kips.

Area of one leg of stirrup =0.11 sq. in.

$$S = \frac{\phi A_v f_y d}{V_u - \phi V_c}$$

$$= \frac{(1.0)(0.22)(60)(10.5)}{20.30} = 6.83$$

Spacing of the #3 stirrups =6.83 in. c/c.

Use a spacing of 4 in. c/c. Use eight, #3 stirrups on either side.

Design of Over-Reinforced Section

From calculation $p_{bal} = 0.04559$

For a over-reinforced section use $p_{max} = 1.5\, p_{bal}$

$= (1.5)(0.04559)$

$= 0.0684$

Area of steel $= P_{max} . b . d = (0.0684)(8)(10.5) = 5.75$ sq. in.

Use six, #9 bars in two rows.

A_s, Area of steel provided =6.00 sq. in.

$$0.5\ f'_c bc = A_s E_s \left(\frac{d-c}{c}\right)\epsilon_u$$

$$(0.5)(10000)(8)(c)=(6.00)(29)(10^6)\left(\frac{10.5-c}{c}\right)(0.0025)$$

C=6.55 sq. in.

$$M_u=(0.5)(f'_c)(b)(c)(d-c/3)$$

$$=(0.5)(10)(8)(6.55)(10.5-(6.55/3))$$

$$=2178.9 \text{ kip-in.}$$

$$=181.580 \text{ kip-ft}$$

P(1/6)=181.5.,P(7/6)=181.5.,

Solving for P, P=155.64 kips.

V_u=77.820 kips

$V_c=\phi 2\sqrt{f'_c}$

In all the shear calculations, the ϕ value is assumed to be equal to 1.

$$V_c=2\sqrt{f'_c}\ bd$$

$$\phi V_c=16.800 \text{ kips}$$

$$V_u-\phi V_c=61.02 \text{ kips}$$

Using #4 bars as stirrups the area of one leg =0.2 sq. in.

Area of two legs of the stirrup= (2)(0.2) =0.4 sq. in.

$$S=\frac{(1.0)(0.4)(60)(10.5)}{61.02}=4.13 \text{ in.}$$

Use #4 stirrups at 3 in. c/c.

Nominal Shear Stress Calculation

Shear Specimen I

Shear span, (a)	=28in.
Compressive strength of concrete	=9500 psi
Oiagonal cracking load	=38000 lb.
Oiagonal cracking shear	=19000 lb.
Nominal cracking shear stress	=19000/(b.d)
b=8 in, d=10.6875 in.	=19000/((10.6875)(8))
	=222 psi.
Ultimate load	=42000 lb.
Ultimate shear	=21000 lb.
Ultimate shear stress	=21000/((8)(10.6875))
	=246 psi.
Ultimate moment= (49)(12000)	=588000 lb-in.
Steel ratio, (A_s/bd)	=0.02105.

Shear force (ACI EQ 11-3) $V_c = 2\sqrt{f'_c}b_w d = (2)(\sqrt{9500})(8)(10.6875)$

=16700 lb.

Shear force (ACI EQ 11-6) $= (1.9\sqrt{f'_c} + 2500p\frac{V_u d}{M_u})(b_w)(d)$

$= 1.9\sqrt{9500} + (2500)(0.02105)((21000)(10.6875)/588000)(b_w)(d)$

=(205)(8)(10.6875)

=17600 lb.

Shear stress calculated using zsutty's equation:

Critial shear force $V_{cr} = 59\ (f'_c p\frac{d}{a})^{0.333}\ (b_w)(d)$

(a=shear span 28 in.)

$=(250)(b_w)(d)$

$=(250)(8)(10.6875)$

$=21400$ lb.

Ultimate shear force $=(63.4\,(f'_c p\frac{d}{a})^{0.333}(b_w)(d)$

$=(269)(b_w)(d)$

$=(269)(8)(10.6875)$

$=23000$ lb.

Shear Stress Calculation for Specimen II

A diagonal shear crack passes through the beam and make the stirrups yield. The number of stirrups yielding is given by the following equation when the angle of the crack is at 45 degrees,

$$N=(\frac{l_{cr}}{S})$$

where l_{cr}=horizontal crack length,

S=spacing of stirrups.

Here l_{cr}=18.75 inches (measured on the beam after failure) and S=11 inches,

$N=(18.75/11)=1.7$

Shear taken by steel $=(N)(A_v)(f_y)$,

where f_y is the yield stress of steel. For the #3 bar, the yield stress is found to be 63.5 ksi (Table 3.1).

Area of #3, Bar=0.11 sq. in.

$=(1.7)(0.22)(63.5)$

$=23.7$ kips

Ultimate load taken by the beam $=86.7$ kips

Ultimate shear taken by the beam =(86.7/2)

=43.35 kips

Shear taken by concrete =43.35-23.7

=19.65 kips

=19650 lb.

Ultimate Shear capacity of concrete =19650/((8)(10.6875))

=230 psi

Ultimate moment= (101)(12000) =1212000 lb-in.

Steel ratio, (A_s/bd) =0.02105

Shear force (ACI EQ 11-3) $V_c = 2\sqrt{f'_c}\, b_w d = (2)\sqrt{(11400)}(8)(10.6875)$

=18300 lb.

Shear force (ACI EQ 11-6) $= (1.9\sqrt{f'_c} + 2500p\dfrac{V_u d}{M_u})(b_w)(d)$

$= 1.9\sqrt{(11400)} + (2500)(0.02105)((43000)(10.6875)/1212000)\ (b_w)(d)$

=(223)(8)(10.6875)

=19000 lb.

Shear stress calculated using zsutty's equation:

Critical shear force $V_{cr} = (59\ (f'_c p \cdot \dfrac{d}{a}))^{0.333}\ (b_w)(d)$

(a=shear span 28 in.)
$(f'_c$ = 11400 psi)
(p = 0.02105)
(d = 10.6875 in.)

$=(266)(b_w)(d)$

=(266)(8)(10.6875)

=22700 lb.

Ultimate shear force $= (63.4\ (f'_c p \dfrac{d}{a}))^{0.333}\ (b_w)(d)$

$=(286)(b_w)(d)$

=(286)(8)(10.6875)

=24400 lb.

Calculation of Modulus of Elasticity

To calculate the secant modulus of elasticity, 0.45 times the ultimate compressive strength was determined. A straight line was drawn connecting that point to the origin in the cylinder stress-strain curve. The slope of the line gave the secant modulus of elasticity. Two cylinder stress-strain curves were used for this purpose. The slope of the line for the first curve was found to be 6.55 X 10^6 psi [45.1 GPa]. The slope of the line for the second curve was found to be 6.21 X 10^6 psi [42.7 GPa]. The mean value of the modulus of elasticity was found to be 6.42 X 10^6 psi [44.2 GPa]. The value calculated by using the ACI formula was,

$$E_c = 33\ W^{1.5}\sqrt{f'_c}$$

where W=dry unit weight of concrete

f'_c=mean ultimate compressive strength of concrete.

The dry unit weight was determined by weighing six 3 in. X 6 in. cylinders and the average value was foud to be 153 pcf (2470 kg/m^3) and the mean ultimate compressive strength was found to be 11860 psi[82.0 MPa].

$$E_c = 33 \times (153)^{1.5} \times \sqrt{11860}$$

$$E_c = 6.78 \times 10^6 \text{ psi}[46.7 \text{ GPa}]$$

The value predicted by the ACI formula was found to be higher than the actual value by nearly 6 percent.

Sample Calculations for the Ultimate Moment

Shear Specimen I

<u>Location of Neutral Axis</u>

To find the absolute strain values at any given load the initial strains (no load strain readings) are subtracted from the corresponding strain values.

Strain readings at	top	2 in. from top	4 in. from top	6 in. from top
Strain at no load:	3	-2	2	3
Strain at ultimate load (91.68 kips)	-543	-221	-94	-26
Absolute strain values	-546	-219	-96	-29

Strain gradient for last 2 in. =(-96-(-29))/2 =-33.5 =-34

Location of neutral axis= 6 +(-29/-34) =6.86 in.(from top)

<u>Triangular Stress Block Moments</u>

For the triangular stress block the actual extreme fiber stress was used to calculate the moment (from Table. 5.19).

$M_u = 0.5\ f_c.c.b.(d-(c/3))$

Where f_c = extreme fiber stress calculated using cylinder stress-strain curve 2 (3430 psi)

b = width of the beam (8 in.)

d = depth of the beam (10.6875 in.)

c = depth of the neutral axis (6.87 in.)

M_u=(0.5)(3430)(6.87)(8)(10.6875-(6.87/3))

= 790000 lb-in.

= 65.8 kip-ft.

Actual moment=(Pl/6)=(42)(7)/6 =49 kip-ft.

Calculated moment =65.8 kip-ft.

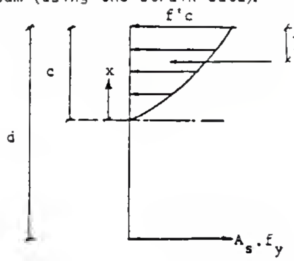
$$\frac{M_{actual}}{M_{calculated}} = \frac{49}{65.8} = 0.74$$

Parabolic Stress Block Moments

Using the strain data the neutral axis was located. The stresses corresponding to the strains are calculated using the cylinder stress strain curve. A second degree equation was fitted to give the stress at any point at a distance x measured from the neutral axis. The equation of the curve was used to get the area of the stress block and the location of the centroid of the stress block by integration. The equation for the under-reinforced beam's stress block at ultimate load is,

f_c=(84.3 x^2 -90 x),

where x is the distance from the neutral axis to the given fiber. The depth of the neutral axis was found to be 6.87 in. from the top of the beam (using the strain data).



The area under the curve is given by integrating between the neutral axis and the top of the beam.

$$A_b = \int_0^{6.87} (84.3\ x^2 - 90\ x)\ dx.$$

On integration,

$$= \left(84.3\ (x^3/3 - 90(x^2/2))\right]_0^{6.87}$$

$$= 6990$$

Location of centroid is given by $= \dfrac{\int Y\ .x.dx}{A_b}$

$$\frac{\int Y.x.dx}{A_b} = \frac{\int_0^{6.87} (84.3\ x^2 - 90\ x).x.dx}{6990}$$

$$= \frac{\left(84.3\ (x^4/4) - 90\ (x^3/3)\right)\Big]_0^{6.87}}{6990}$$

$$= 5.32 \text{ in.}$$

Centroidal distance from top $\bar{X}$ =(6.87-5.32)

=1.55 in.

Total moment capacity of the beam =Ab.b.(d-$\bar{X}$)

=(6990)(8)((10.6875-1.55)

=510000 lb-in.

=42.5 kip-ft.

Actual moment capacity of the beam =49 kip-ft.

calculated moment capacity of the beam =42.5 kip-ft.

$$\frac{M_{actual}}{M_{calculated}} = \frac{49}{42.5} = 1.15$$

Shear Specimen II

Location of Neutral Axis

To find the absolute strain values at any given load the initial strains (no load strain readings) are subtracted from the corresponding strain values.

Strain readings at	top	2 in. from top	4 in. from top
Strain at no load:	9	1	-1
Strain at ultimate load (91.68 kips)	-1138	-539	-40
Absolute strain values	-1147	-540	-39

Strain gradient for top 2 inches $=(-1147-(-540))/2$ $=-303.5$
$=-304\ \mu$ in./in.

Strain gradient for next 2 inches $=(-540-(-39))/2$ $=-250.5$
$=-251\ \mu$ in./in.

Average strain gradient $=(-304+(-251))/2$

$=277.5\ \mu$ in./in.

$=278\ \mu$ in./in.

Location of neutral axis= $4 + (-39/-278)$ $=4.14$ in.(from top)

Triangular Stress Block Moments

For the triangular stress block the actual extreme fiber stress was used to calculate the moment (from Table. 5.22)

$$M_u = 0.5\ f_c.c.b.(d-(c/3))$$

where f_c = average cylinder compressive strength (7100 psi),

$B_1 = 0.65$,

b = width of the beam (8 in.),

d = depth of the beam (10.6875 in.),

c = depth of the neutral axis (2.12 in.).

M_u=(0.5)(7100)(4.14)(8)(10.6875-(4.14/3))

= 1094000 lb-in.

= 91.2 kip-ft.

Actual moment=(Pl/6)=(86.7)(7)/6 =101 kip-ft.

Calculated moment =91.2 kip-ft.

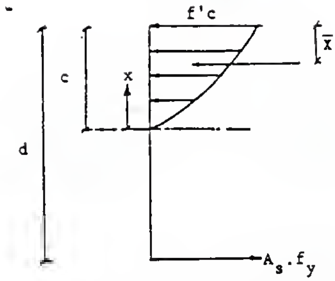
$$\frac{M_{actual}}{M_{calculated}} = \frac{101}{91.2} = 1.11$$

<u>Parabolic Stress Block Moments</u>

Using the strain data the neutral axis was located. The stresses corresponding to the strains were calculated using the cylinder stress strain curve. A second degree equation was fitted to give the stress at any point at a distance x measured from neutral axis. The equation of the curve was used to get the area of the stress block and the location of the centroid of the stress block by integration. The equation for the under-reinforced beam's stress block at ultimate load is,

f_c=(63.7 x^2 + 1451 x),

where x is the distance from the neutral axis to the given fiber. The depth of the neutral axis was found out to be 4.14 in. from the top of the beam (using the strain data).



The area under the curve is given by integrating between the the neutral axis and the top of the beam.

$$A_b = \int_0^{4.14} (63.7\ x^2 + 1451\ x)\ dx.$$

on integration,

$$= \left(63.7\ (x^3/3 + 1451\ (x^2/2))\right]_0^{4.14}$$

$$= 14000$$

Location of centroid is given by $= \dfrac{\int Y\ x.dx}{A_b}$

$$\frac{\int Y\ x.dx}{A_b} = \frac{\int_0^{4.14} (63.7\ x^2 + 1451\ x).x.dx}{14000}$$

$$= \frac{\left(63.7\ (x^4/4) + 1451\ (x^3/3))\right]_0^{4.14}}{14000}$$

$$= 2.79 \text{ in.}$$

Centroidal distance from top $\quad \overline{X} = (4.14-2.79)$

$= 1.35$ in.

Total moment capacity of the beam $= Ab.b.(d-\overline{X})$

$= (14000)(8)((10.6875-1.35)$

$= 1050000$ lb-in.

$= 87.5$ kip-ft.

Actual moment capacity of the beam =101 kip-ft.

calculated moment capacity of the beam =87.5 kip-ft.

$$\frac{M_{actual}}{M_{calculated}} = \frac{101}{87.5} = 1.15$$

Under-Reinforced Beam

Location of neutral axis

To find the absolute strain values at any given load the initial strains (no load strain readings) are subtracted from the corresponding strain values.

Strain readings at	top	2 in. from top	4 in. from top
Strain at no load:	4	1	-3
Strain at ultimate load (91.68 kips)	-2413	-141	502
Absolute strain values	-2417	-142	505

Strain gradient =(-142-(505))/2 =-323.5

=-324 μ in./in.

Location of neutral axis= 2 +(-142/-324) =2.44 in.(from top)

Rectangular Stress Block Moments

$$M_u = 0.85\ f'_c.(B_1.c).b.(d-(B_1c/2)$$

where f'_c = average cylinder compressive strength (11700 psi),

B_1 = 0.65,

b = width of the beam (8 in.),

d = depth of the beam (10.6875 in.),

c = depth of the neutral axis (2.44 in.).

M_u=(0.85)(11700)((0.65)(2.44))(8)(10.6875-((0.65)(2.44))/2)

= 1250000 lb-in.

= 104 kip-ft.

Actual moment=(Pl/6)=(91.7)(7)/6 =107 kip-ft.

Calculated moment =104 kip-ft.

$$\frac{M_{actual}}{M_{calculated}} = \frac{107}{104} = 1.03$$

Triangular Stress Block Moments

For the triangular stress block the actual extreme fiber stress was used to calculate the moment (from Table. 5.25)

M_u= 0.5 f'_c.c.b.(d-(c/3))

where f_c = extreme fiber stress calculated using cylinder stress-strain curve 2 (12200 psi),

b = width of the beam (8 in.),

d = depth of the beam (10.6875 in.),

c = depth of the neutral axis (2.44 in.).

M_u=(0.5)(12200)(2.44)(8)(10.6875-(2.44/3))

= 1180000 lb-in.

= 98.0 kip-ft.

Actual moment=(Pl/6)=(91.7)(7)/6 =107 kip-ft.

Calculated moment =98.0 kip-ft.

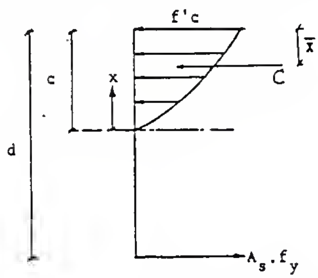
$$\frac{M_{actual}}{M_{calculated}} = \frac{107}{98.0} = 1.09$$

Parabolic Stress Block Moments

Using the strain data the neutral axis was located. The stresses corresponding to the strains were calculated using the cylinder stress strain curve. A second degree equation was fitted to give the stress at any point at a distance x measured from the neutral axis. The equation of the curve was used to get the area of the stress block and the location of the centroid of the stress block by integration. The equation for the under reinforced beam's stress block at ultimate load is,

$f_c = (1486.3\ x^2 + 1379.7\ x),$

where x is the distance from the neutral axis to the given fiber. The depth of the neutral axis was found out to be 2.44 in. from the top of the beam (using the strain data).



The area under the curve is given by integration between the neutral axis and the top of the beam.

$$A_b = \int_0^{2.44} (1486.3\ x^2 + 1379.7\ x)\ dx.$$

On integration,

$$A_b = (1486.3\ (x^3/3 + 1379.7\ (x^2/2))\Big]_0^{2.44}$$

$$= 11300.$$

Location of the centroid is given by $= \dfrac{Y\ x.dx}{A_b}$

$$\frac{Y.x.dx}{A_b} = \frac{\int_0^{2.44} (1486.3\ x^2 + 1379.7\ x).x.dx}{11300}$$

$$= \frac{\left.(1486.3\ (x^4/4) + 1379.7\ (x^3/3))\right]_0^{2.44}}{11300}$$

$= 1.76$ in.

Centroidal distance from top $\bar{X}$ $=(2.44-1.76)$

$=0.68$in.

Total moment capacity of the beam $=Ab.b.(d-\bar{X})$

$=(11300)(8)((10.6875-0.68)$

$=905000$ lb-in.

$=75.4$ kip-ft.

Actual moment capacity of the beam $=107$ kip-ft.

calculated moment capacity of the beam $=75.4$ kip-ft.

$$\frac{M_{actual}}{M_{calculated}} = \frac{107}{75.4} = 1.42$$

Over-Reinforced Beam

<u>Location of Neutral Axis</u>

To find the absolute strain values at any given load, the initial strains (no load strain readings) are subtracted from the corresponding strain values.

Strain readings at	top	2 in. from top	4 in. from top	6 in. from top
Strain at no load:	-11	1	0	0

Strain at ultimate load (180.0 kips)	-2604	-1696	-626	1146
Absolute strain values	-2587	-1697	-626	1146

Strain gradient for last 2 in. =(-626-(1146))/2 =-886

Strain gradient =-886 U in./in.

Location of neutral axis= 4 +(-626/-886) =4.71 in.(from top)

Rectangular Stress Block Moments

M_u= 0.85 f'_c.(B_1.c).b.(d-(B_1/2)

where f'_c = average cylinder compressive strength (12100 psi)

B_1 = 0.65,

b = width of the beam (8 in.),

d = depth of the beam (9.3125 in.),

c = depth of the neutral axis (4.71 in.).

M_u=(0.85)(12100)((0.65)(4.71))(8)(9.3125-((0.65)(4.71))/2)

= 1960000 lb-in.

= 163 kip-ft.

Actual moment=(Pl/6)=(180.0)(7)/6 =210.0 kip-ft.

Calculated moment =163 kip-ft.

$$\frac{M_{actual}}{M_{calculated}} = \frac{210.0}{163} = 1.29$$

Triangular Stress Block Moments

For the triangular stress block the actual extreme fiber stress was used to culate the moment (from Table. 5.30).

M_u= 0.5 f'_c.c.b.(d-(c/3)

wheref_c = extreme fiber stress calculated using cylinder stress-strain curve 2 (12400 psi),

b = width of the beam (8 in.),

d = depth of the beam (9.3125 in.),

c = depth of the neutral axis (4.71 in.).

M_u=(0.5)(12400)(4.71)(8)(9.3125-(4.71/3))

= 1800000 lb-in.

= 150.0 kip-ft.

Actual moment=(Pl/6)=(180.0)(7)/6	=210.0 kip-ft.
Calculated moment	=150.0 kip-ft.

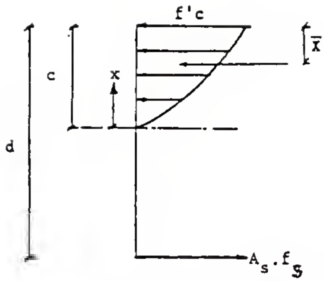
$$\frac{M_{actual}}{M_{calculated}} = \frac{210.0}{150.0} = 1.4$$

<u>Parabolic Stress Block Moments</u>

Using the strain data the neutral axis was located. The stresses corresponding to the strains were calculated using the cylinder stress strain curve. A second degree equation was fitted to give the stress at any point at a distance x measured from neutral axis. The equation of the curve was used to get the area under the curve and the location of the centroid of the curve by integration. The equation for the under reinforced beam's stress block at ultimate load is,

$Y(x)=(-568.7\ x^2 + 5296\ x)$

where x is the distance from the neutral axis to the given fiber. The depth of neutral axis was found to be 4.71 in. from the top of the beam (using the strain data).



The area under the curve is given by integration between the neutral axis and the top of the beam.

$$A_b = \int_0^{4.71} (-568.7\ x^2\ x + 5296.3\ x)\ dx$$

On integration,

$$A_b = (-568.7)(x^3/3) + (5296.3)((x^2/2))\Big]_0^{4.71}$$

$$=39000$$

Location of centroid from neutral axis is given by $= \dfrac{\int Y\ x.dx}{A_b}$

$$\frac{\int Y\ x.dx}{A_b} = \frac{\int_0^{4.71} (-568.7\ x^2 + 5296.3x).x.dx}{39000}$$

$$= \frac{(-568.7\ (x^3/3) + 5296.3\ (x^2/2))\Big]_0^{4.71}}{39000}$$

$$= 2.94 \text{ in.}$$

Centroidal distance from top $\overline{X} = (4.71-2.94)$

$= 1.77$ in.

Total moment capacity of the beam $= A_b.b.(d-\overline{X})$

$= (39000)(8)(9.3125-1.77)$

$= 2350000$ lb-in.

=196 kip-ft.

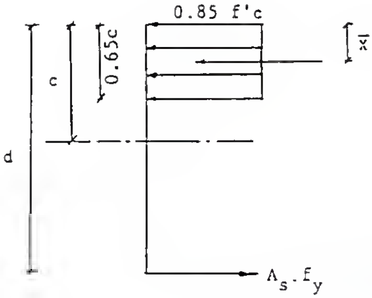
Actual moment capacity of the beam =210 kip-ft.

Calculated moment capacity of the beam =196 kip-ft.

$$\frac{M_{actual}}{M_{calculated}} = \frac{210}{196} = 1.07$$

Prediction of the Ultimate Moment Capacity

Under-Reinforced Beam



Rectangular Stress Block

$a=B_1c$ where $B_1=0.65$

$d=10.6875$ in. $f'_c=11700$ psi

As=1.8 sq. in. (three, #7 bars)

$f_y=64700$ psi

b=8 in.

$C=0.85\ f'_c ab$

$T=As.f_y$

Equating C=T,

$$a=\frac{(1.8)(64700)}{(0.85)(11700)(8)} = 1.46 \text{ in.}$$

$a=B_1c$ where $B_1=0.65$

$\therefore$ c=(1.46/0.65)=2.25 in.

Depth of neutral axis (calculated) =2.25 in.

Depth of neutral axis (Test result) =2.44 in.

Ultimate moment $=0.85f'_c.a.b.(d-a/2)$

$=(0.85)(11700)(1.46)(8)(9.5625)$

=1110000 lb-in.

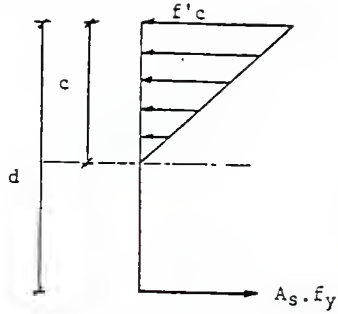
Ultimate moment =92.6 kip-ft.

Actual moment taken by the beam =107 kip-ft.

$$\frac{M_{actual}}{M_{calculated}} = \frac{107}{92.6} = 1.16$$

Triangular Stress Block

Properties of the triangular stress block;



d=10.6875 in.

b=8 in.

f'_c=11700 psi

As=1.8 sq. in.

f_y=64700 psi

C=0.5 f'_c.c.b.

T=As.f_y

Equating C=T,

$$c=\frac{(1.8)(64700)}{(0.5)(11700)(8)}=2.49 \text{ in.}$$

Calculated depth of neutral axis. =2.49 in.

Actual depth of neutral axis. =2.44 in.

Moment capacity calculated,

$$M_u = (0.5)(f'_c)(c)(b)(d-a/3)$$

$$= (0.5)(11700)(2.49)(8)(10.6875-(2.49/3))$$

$$= 1150000 \text{ lb-in.}$$

$$= 96 \text{ kip-ft.}$$

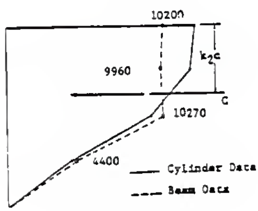
Actual moment =107 kip-ft.

Calculated moment =96 kip-ft.

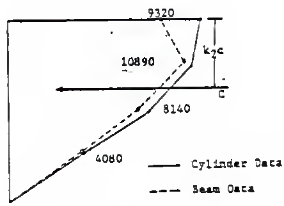
$$\frac{M_{actual}}{M_{Calculated}}=\frac{106.96}{95.73}=1.11$$

<u>Parabolic Stress Block</u>

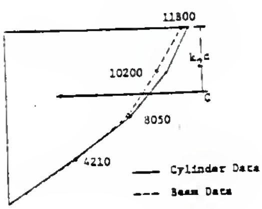
The shape of the stress block suggested by Ref. 14 is used to predict the ultimate moment capacity of the beam. The stress blocks at ultimate load for all the four beams tested in Ref. 14 were plotted. The average stresses at various depth were calculated. A parabolic curve was fitted through these points using the least sqaure method to give the stress at any point x measured from the neutral axis.



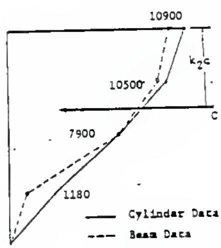
Beam A



Beam B



Beam C



Beam D

Using this equation the area under the stress block and the location of the centroid were calculated. The equation is,

$fc = (-9988.1\ (x/c)^2 + 20828.7\ (x/c))$

$$Ab = \int_0^c (-9988.1\ (x/c)^2 + 20828.7\ (x/c))$$
$$= (-9988.1\ (x^3/3c^2) + 20828.7\ (x^2/2c))\Big]_0^c$$

Total compressive force C,

$$C = (8)(-9988.1\ (x^3/3c^2) + 20828.7\ (x^2/2c))\Big]_0^c$$

This area is equal to $As.f_y$.

$$As.f_y = (8)((-9988.1\ (c/3) + 20828.7\ (c/2))$$

$$(1.8)(64700) = (8)(7085)(c).$$

Solving for c,

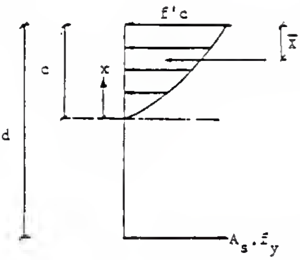
$c = 2.05$ in.

Calculated depth of neutral axis =2.05 in.

Actual depth of neutral axis. =2.44 in.

Area under the curve A_b =14500

Location of the centroid,



$$X = \frac{\int_0^{2.05} Y_{(x)}.x.dx}{A_b}$$

$$= \frac{\int_0^{2.05} (-9988.1(x^3/c^2) + 20828(\ x^2/c).dx}{14500}$$

$= 1.29$ in.

Centroidal distance from the top $\overline{X}$ $=(2.05-1.29)$

$=0.76$ in.

Moment capacity of the beam $=A_b.b.(d-\overline{X})$

$=(14500)(8)(10.6875-0.76)$

=1150000 lb-in.

=95.8 kip-ft.

Actual moment capacity of the beam =107 kip-ft.

$$\frac{M_{actual}}{M_{calculated}} = \frac{107}{95.8} = 1.11$$

Over-Reinforced Beam

Rectangular Stress Block

$a=B_1c$ where $B_1=0.65$

$d=9.3125$ in. $f'_c=12100$ psi

$As=6.0$ sq. in.

$f_y=63800$ psi $Es=(29)(10^6)$

$b=8$ in.

$C=0.85\ f'_c ab$

$T=As.f_s$, where $f_s=E_s.\epsilon_s$

strain diagram:

An ultimate concrete strain value of 0.0025 in./in. is assumed for calculating the location of the neutral axis.

$$\frac{\epsilon_s}{(d-c)} = \frac{\epsilon_u}{c}$$

$$\epsilon_s = \frac{\epsilon_u(d-c)}{c}$$

$As.fs = 0.85.f'_c.(B_1c).b$

$As.Es.\epsilon s=0.85.f'_c.(B_1c).b$

$As.Es.(\epsilon_u(d-c)/c)=(0.85)(12100)(0.65)(c)(8)$

$(6)(29)(10^6)(0.0025)((9.3125-c)/c)=(0.85)(12100)(0.65)(c)(8)$

Solving for c,

$c=5.53$ in.

Depth of neutral axis (calculated) $=5.53$ in.

Depth of neutral axis (actual) $=4.71$ in.

Ultimate moment,

$$M_u=(0.85)(12100)((0.65)(5.53))(8)(9.3125-(0.65)(5.53/2)))$$

$=2220000$ lb.-in.

$=185$ kip-ft

Actual moment taken by the beam $=210$ kip-ft.

Calculated moment $=185$ kip-ft.

$$\frac{M_{actual}}{M_{calculated}}=\frac{210}{185}=1.14$$

<u>Triangular Stress Block</u>

$d=9.3125$ in.

$b=8$ in.

$f'_c=12100$ psi

$As=6.0$ sq. in.

$f_y=63800$ psi

$C=0.5\ f'_c.c.b.$

$T=As.f_s$

strain diagram:

An ultimate concrete strain value of 0.0025 in./in. is assumed for calculating the location of the neutral axis, c measured from the top of the beam.

$$\frac{\epsilon_s}{(d-c)}=\frac{\epsilon_u}{c}$$

$$\epsilon_s = \frac{\epsilon_u(d-c)}{c},$$

$A_s.f_s=0.5.f_c'.c.b.$

$A_s.E_s.\epsilon_s=0.5.f_c'.c.b$

$A_s.E_s.(\epsilon_u(d-c)/c)=(0.5)(12100)(c)(8)$

$(6)(29)(10^6)(0.0025)((9.3125-c)/c)=(0.5)(12100)(c)(8)$

Solving for c,

c=5.70 in.

Depth of neutral axis (calculated) =5.70 in.

Depth of neutral axis (actual) =4.71 in.

Ultimate moment,

$M_u=(0.5)(12100)(5.70)(8)(9.3125-(5.70/3))$
=2040000 lb-in.

=170 kip-in.

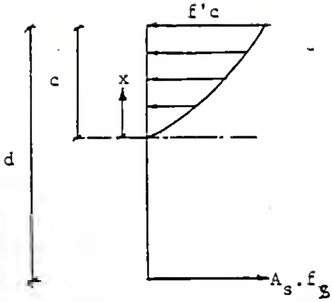
Actual moment taken by the beam =210 kip-ft.

Calculated moment =170 kip-ft.

$$\frac{M_{actual}}{M_{calculated}} = \frac{210}{170} = 1.24$$

Parabolic Stress Block

The shape of the stress block suggested by Ref. 14 is used to predict the ultimate moment capacity of the beam. The stress block at ultimate load for all the four beams tested in Ref. 14 was plotted. The average stresses at various depth were calculated. A parabolic curve was fitted through these points using the least squares method to give the stress at any point x measured from the neutral axis.



Using this equation, the area under the stress block and the location of the centroid are calculated. The equation is,

$$Y_{(x)} = (-9988.1\ (x/c)^2 + 20828.7\ (x/c))$$

$$A_b = \int_0^c (-9988.1\ (x/c)^2 + 20828.7\ (x/c))\ dx$$

$$= (-9988.1\ (x^3/3c^2 + 20828.7\ (x^2/2c) \Big|_0^c$$

Total compressive force C,

$$C = (8)(-9988.1(x^3/3c^2) + 20828.7\ (x^2/2c) \Big|_0^c$$

$$As.f_s = (b)(-9988.1\ (c/3) + 20828.7\ (c/2))$$

$$As.f_s = (8)(7085)(c) = 56700(c)$$

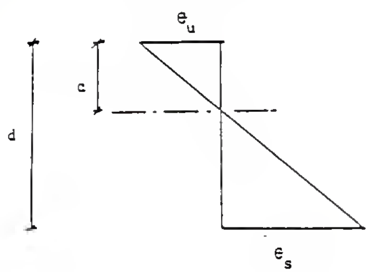
strain diagram:

An ultimate concrete strain value of 0.0025 in./in. is assumed for calculating the location of the neutral axis.

$$\frac{\epsilon_s}{(d-c)} = \frac{\epsilon_u}{c}$$

$$\epsilon s = \frac{\epsilon_u(d-C)}{c}$$

As.fs=56680.0 c



$$As.Es.\left(\frac{\epsilon_u(d-c)}{c}\right)=56700c$$

$$(6.0)(29)(10^{6})(0.0025(9.3125-c)/c)=56700c$$

$$c^2 + 7.675c - 71.47=0$$

Solving for c,

c=5.45 in.

Oepth of neutral axis (calculated) =5.45 in.

Oepth of neutral axis (actual) =4.71 in.

Area under the curve $Ab=((-9988.1/3)+(20828/2))(5.45)$

$Ab=38600.$

Location of the centroid from neutral axis,

$$C.G= \frac{\int_0^c Y_{(x)}.x.dx}{A_b}$$

$$= \frac{\int_0^c (-9988.1\,(x^3/c^2) + 20828\,(x^2/c).x.dx}{38600}$$

$$= \frac{(-9988.1\,((x^4/4)/c^2) + 20828\,((x^3/3)/c)]_0^c}{38600}$$

$$= \frac{[(-9988.1\,(c^2/4) + (20828\,(c^2/3)]}{38600}$$

=3.42 in.

Substituting the value of c in the equation,

Centroidal distance from neutral axis =5.45 in.

Centroidal distance from the top $\overline{X}$ =(5.45-3.42) =2.03 in.

The ultimate moment capacity =(38600)(8)(9.3125-2.03)

=2250000 lb-in.

=188 kip-ft.

Actual moment taken by the beam =210 kip-ft.

Calculated moment =188 kip-ft.

$$\frac{M_{actual}}{M_{calculated}} = \frac{210}{188} = 1.12$$

APPENDIX III

BASIC PROGRAMS

Program for Shear Specimen 1

```
10 REM SHEAR SPECIMEN 1  NAME alishl
20 REM LPRINT "RESULT OF THE TEST DATA FOR SHEAR SPECIMEN-1"
30 REM LPRINT "NO SHEAR REINFORCEMENT ALISHI"
50 LPRINT :LPRINT :LPRINT :LPRINT :LPRINT :
60 DIM W(30),ST(30),S2(30),S4(30),S6(30),STRO(30),STR1(30),STR2(30),
   LEV(3,30),NAT(30),SA1(30),SA2(30),SA3(30),SA0(30),STR3(30),UN(30),
   COMP(3,30),AB(30),CM(3,30),AM(30),ACCU(3,30),ER(3,30),X(30),AS(30),Y(30),
   IC(30),YS(30),IS(30),ICCR(30)
70 DIM ISCR(30),ITCR(30),MCR(30),TI(30),IE(30),DEFC(30),ACCD(30),IT(30),
   DEFA(30),MA(30),A1(30),A2(30),A3(30),NX(30),YT(30),A(3,4),XX(3),TTX(30),
   TAX(30),TAT(30),XBAR(30),TAY(30)
80 N=22
90 FOR I=1 TO N
100 READ DEFA(I),W(I),ST(I),S2(I),S4(I),S6(I)
110 DATA 0,0,3,-2,2,3
120 DATA 68,2970,-22.2,-20.75,-6.25,1.45
130 DATA 90,5925,-45.45,-16.85,-12.05,4.8
140 DATA 98,8965,-72.05,-28.4,-19.8,5.3
150 DATA 109,11865,-103.5,-42.5,-29.5,13.55
160 DATA 117,14835,-134,-70.6,-34.3,18.85
170 DATA 129,17665,-174.15,-80.6,-38.2,31.85
180 DATA 143,20645,-231.75,-115.6,-40.15,22.2
190 DATA 143,20000,-230.75,-117.05,-42.55,25.1
200 DATA 161,23715,-284.0,-136.85,-44.5,36.7
210 DATA 177,26690,-329.45,-141.75,-53.5,23.2
220 DATA 192,29700,-374.0,-163.6,-62.7,7.7
230 DATA 209,32710,-419.95,-203.65,-69.6,4.8
240 DATA 218,34800,-447.05,-216.25,-73.5,-4.8
250 DATA 226,36555,-473.1,-229.8,-79.3,-5.8
260 DATA 234,38015,-491.55,-215.75,-79.3,-9.6
270 DATA 243,39790,-515.75,-226.9,-83.2,-11.6
280 DATA 661,42000,-540.45,-239,-88,-16.4
290 DATA 661,37410,-492.55,-184.8,-75.4,-22.2
300 DATA 661,37190,-491.055,-189.15,-73.5,-24.1
310 DATA 661,38200,-503,-192.05,-79.3,-23.2
320 DATA 668,42000,-543.3,-220.6,-93.8,-26.1
330 SA1(I)=(S2(I)-S2(1))
340 SA2(I)=(S4(I)-S4(1))
350 SA3(I)=(S6(I)-S6(1))
360 SA0(I)=(ST(I)-ST(1))
370 NEXT I
390 LPRINT "LOAD IN NUTERAL AXIS  STRESS AT  STRESS AT   STRESS AT  STRESS AT"
400 LPRINT "  KIPS.   DEPTH IN       TOP          2 IN.       4 IN         6 IN."
410 FOR I=2 TO N
420 IF SGN(SA3(I))=-1 THEN GOTO 450
430 UN(I)=(((SA0(I)-SA1(I))/2)+(SA1(I)-SA2(I))/2)/2:PRINT "UN(";I;")=";UN(I)
440 NAT(I)=(4+(SA2(I)/UN(I))):PRINT"NAT(";I;")=";NAT(I):GOTO 470
450 UN(I)=(SA2(I)-SA3(I))/2
460 NAT(I)=(4+(SA2(I)/UN(I)))
470 AA=1:BB=1:CC=1:DD=1
480 IF SGN(SA1(I))=1 THEN AA=0
490 IF SGN(SA2(I))=1 THEN BB=0
500 IF SGN(SA3(I))=1 THEN CC=0
510 IF SGN(SA0(I))=1 THEN DD=0
520 SA1(I)=ABS(SA1(I))
530 SA2(I)=ABS(SA2(I))
540 SA3(I)=ABS(SA3(I))
```

```
550 SAO(I)=ABS(SAO(I))
560 STRO(I)=(-4.06906E-07*(SAO(I)^3)+5.619357E-04*(SAO(I)^2)+6.060692*(SAO(I))+
    18.562801#)
570 STR1(I)=(-4.06906E-07*(SA1(I)^3)+5.619357E-04*(SA1(I)^2)+6.060692*(SA1(I))+
    18.562801#)
580 STR2(I)=(-4.06906E-07*(SA2(I)^3)+5.619357E-04*(SA2(I)^2)+6.060692*(SA2(I))+
    18.562801#)
590 STR3(I)=(-4.06906E-07*(SA3(I)^3)+5.619357E-04*(SA3(I)^2)+6.060692*(SA3(I))+
    18.562801#)
600 IF AA=0 THEN STR1(I)=-STR1(I)
610 IF BB=0 THEN STR2(I)=-STR2(I)
620 IF CC=0 THEN STR3(I)=-STR3(I)
630 LPRINT
640 W(I)=W(I)/1000
641 DVN=10
642 STRO(I)=(CINT(STRO(I)/DVN))*DVN
643 STR1(I)=(CINT(STR1(I)/DVN))*DVN
644 STR2(I)=(CINT(STR2(I)/DVN))*DVN
645 STR3(I)=(CINT(STR3(I)/DVN))*DVN
650 LPRINT USING"  ###.#      #.##        #####      #####        ######      #####
    ";W(I),NAT(I),          STRO(I),STR1(I),STR2(I),STR3(I)
660 W(I)=W(I)*1000
670 NEXT I
680 LPRINT CHR$(12)
690 D=10.6875
700 FC=9500
710 B=.65:WB=8
720 FOR COUNTER=0 TO 2
760 FOR I=2 TO N
770 AB(I)=(NAT(I)*B)
780 IF COUNTER=2 GOTO 890
790 IF COUNTER=1 GOTO 820
800 COMP(COUNTER,I)=(STRO(I)*AB(I)*WB)
810 IF COUNTER=0 GOTO 830
820 COMP(COUNTER,I)=(.5*STRO(I)*NAT(I)*WB)
830 PRINT "COMP(";COUNTER,I;")=";COMP(COUNTER,I)
840 IF COUNTER=1 GOTO 870
850 LEV(COUNTER,I)=(D-(AB(I)/2))
860 IF COUNTER=0 GOTO 1290
870 LEV(COUNTER,I)=(D-(NAT(I)/3))
880 IF COUNTER=0 OR 1 GOTO 1290
890 REM PARABALOIC CURVE FITTING
900 NP=3
910 IF NAT(I)>=4 THEN NP=4
920 X(1)=NAT(I)
930 X(2)=(NAT(I)-2)
940 IF NAT(I)<4 THEN X(3)=0:Y(3)=0:GOTO 960
950 X(3)=NAT(I)-4:X(4)=0:Y(3)=STR2(I):Y(4)=0
960 Y(1)=STRO(I)
970 Y(2)=STR1(I)
980 SX=0:SX2=0:SX3=0:SX4=0:SY=0:SXY=0:SX2Y=0
990 FOR T=1 TO NP
1000 SX=SX+X(T)
1010 SY=SY+Y(T)
1020 SX2=SX2+(X(T)^2)
1030 SX3=SX3+(X(T)^3)
1040 SX4=SX4+(X(T)^4)
1050 SXY=SXY+X(T)*Y(T)
1060 SX2Y=SX2Y+(X(T)*X(T)*Y(T))
1070 NEXT T
```

```
1090 A(2,2)=SX2:A(2,3)=SX3
1100 A(3,2)=SX3:A(3,3)=SX4
1110 A(2,4)=SXY:A(3,4)=SX2Y
1120 GOSUB 1980
1130 A1(I)=XX(1)
1140 A2(I)=XX(2)
1150 A3(I)=XX(3)
1160 NX(I)=NAT(I):PRINT ;"A1(";I;")=";A1(I),A2(I),A3(I)
@1170 TTX(I)=NX(I)/10
1180 FOR Z=1 TO 11
1190 TAX(Z)=NX(I)-((Z-1)*TTX(I))
1200 TAY(Z)=(A3(I)*(TAX(Z)^2))+(A2(I)*TAX(Z))+A3(I)
1210 NEXT Z
1220 FOR Z=1 TO 10
1230 TAT(I)=TAT(I)+(TTX(I)*((TAX(Z)*TAY(Z))+((TAY(Z)+TAY(Z+1))*(TAX(Z)+
     TAX(Z+1)))+(TAX(Z+1)*TAY(Z+1))))/6
1240 NEXT Z
1250 AS(I)=(A3(I)*((NX(I)^3)/3))+(A2(I)*((NX(I)^2)/2))+(A1(I)*NX(I))
1260 XBAR(I)=TAT(I)/AS(I):XBAR(I)=NX(I)-XBAR(I)
1270 LEV(COUNTER,I)=(D-XBAR(I))
1280 COMP(COUNTER,I)=(AS(I)*8):PRINT "COMP(";COUNTER,I;")="COMP(COUNTER,I)
1290 PRINT "LEV(";COUNTER,I;")=";LEV(COUNTER,I)
1300 CM(COUNTER,I)=COMP(COUNTER,I)*LEV(COUNTER,I):L=84
1310 AM(I)=(W(I)*L)/6
1320 PRINT "CM(";COUNTER,I;")="CM(COUNTER,I),"AM(";I;")=";AM(I)
1330 ACCU(COUNTER,I)=(AM(I)/CM(COUNTER,I))
1340 ER(COUNTER,I)=((CM(COUNTER,I)-AM(I))/AM(I))*100
1350 PRINT "ACCU(";COUNTER,I;")=";ACCU(COUNTER,I):PRINT :PRINT "ER(";
     COUNTER,I;")=";ER(COUNTER,I)
1360 AM(I)=AM(I)/12000:CM(COUNTER,I)=(CM(COUNTER,I)/12000)
1370 NEXT I
1380 NEXT COUNTER
1390 FOR I=1 TO N
1400 W(I)=W(I)/1000:DEFA(I)=DEFA(I)/1000
1410 NEXT I
1420 FOR COUNT=1 TO 3
1430 LPRINT :LPRINT :LPRINT :LPRINT :LPRINT :
1440 IF COUNT=1 THEN LPRINT "LOAD NO.  LOAD IN  Mu (test)  Mu ([]calc)
     Mu(^calc)  Mu (test)     Mu(test)
1450 IF COUNT=2 THEN LPRINT "LOAD NO.  LOAD IN  Mu (test)  Mu (^ calc)
     Mu(¯calc)  Mu (test)     Mu(test)
1460 IF COUNT=3 THEN LPRINT "LOAD NO.  LOAD IN  Mu (test)  Mu ([]calc)
     Mu(¯calc)  Mu (test)     Mu(test)
1470 IF COUNT=1 THEN LPRINT "            KIPS      KIP-FT      KIP-FT
     KIP-FT     Mu ([]calc)    Mu(^calc)
1480 IF COUNT=2 THEN LPRINT "            KIPS      KIP-FT      KIP-FT
     KIP-FT     Mu (^ calc)    Mu(¯calc)
1490 IF COUNT=3 THEN LPRINT "            KIPS      KIP-FT      KIP-FT
     KIP-FT     Mu ([]calc)    Mu(¯calc)
1500 LPRINT
1510 FOR I=N TO N
1520 IF COUNT=1 THEN LPRINT USING "   ##      ###.#     ###.#      ###.#   ###.#
     ###.##          ###.##";I,W(I),AM(I),CM(0,I),CM(1,I),ACCU(0,I),ACCU(1,I)
1530 NEXT I
1540 FOR I=2 TO N
1550 IF COUNT=2 THEN LPRINT USING "   ##      ###.#     ###.#      ###.#   ###.#
     ###.##          ###.##";I,W(I),AM(I),CM(1,I),CM(2,I),ACCU(1,I),ACCU(2,I)
1560 IF I<N THEN GOTO 1580
1570 IF COUNT=3 THEN LPRINT USING "   ##      ###.#     ###.#      ###.#  ###.#
     ###.##          ###.##";I,W(I),AM(I),CM(0,I),CM(2,I),ACCU(0,I),ACCU(2,I)
```

```
1580 IF COUNT=2 THEN LPRINT
1590 NEXT I
1600 LPRINT CHR$(12)
1610 NEXT COUNT
1620 FOR COUNTER=0 TO 2
1630 FOR I=1 TO N
1640 CM(COUNTER,I)=(CM(COUNTER,I)*12000)
1650 NEXT I
1660 NEXT COUNTER
1670 LPRINT :LPRINT :LPRINT :LPRINT :
1680 LPRINT "LOAD NO.  LOAD IN  DEF ACTUAL    DEF CAL  DEF ACTUAL"
1690 LPRINT "            KIPS         IN.          IN.   DEF CAL"
1700 FOR I=2 TO 18
1710 IF STRO(I)<6700 THEN EC=6216845!
1720 IF STRO(I)>6700 AND STRO(I)<9700 THEN EC=5467505!
1730 IF STRO(I)>9700 THEN EC=3828919!
1740 TD=12:WB=8:AS=1.8:ES=2.9E+07:A=96
1750 MA(I)=AM(I)
1760 YT(I)=(6-NAT(I))
1770 IC(I)=(WB*(TD^3))/12+(A*(YT(I)^2))
1780 YS(I)=(D-NAT(I))
1790 IS(I)=(((ES/EC)-1)*AS)*(YS(I)^2)
1800 IT(I)=IC(I)+IS(I):PRINT "ITCG=";IT(I)
1810 ICCR(I)=(WB*(NAT(I)^3)/12)+((WB*NAT(I))*((NAT(I)/2)^2))
1820 NS=(ES/EC)
1830 ISCR(I)=(NS*AS)*(YS(I)^2)
1840 ITCR(I)=ICCR(I)+ISCR(I):PRINT "ITCR=";ITCR(I)
1850 FCR=7.5*SQR(FC)
1860 MCR(I)=(FCR*IT(I))/(TD-NAT(I))
1870 TI(I)=((MCR(I)/(106.96*12000))^3)
1880 IE(I)=(TI(I)*IT(I))+((1-TI(I))*ITCR(I))
1890 REM IF (AM(I)*12000)<MCR(I) THEN IE(I)=IT(I)
1900 PRINT "IE=";IE(I)
1910 DEFC(I)=(23*W(I)*(84^3)*1000)/(1296*EC*IE(I))
1920 ACCD(I)=(DEFA(I)/DEFC(I))
1930 LPRINT
1940 LPRINT USING "  ##       ##.##    #.###          #.###      ##.##";I,
     W(I),DEFA(I),DEFC(I),ACCD(I)
1950 NEXT I
1960 LPRINT CHR$(12)
1970 END
1980 NS=3
1990 FOR K=1 TO NS
2000 C=A(K,K)
2010 FOR J=K TO (NS+1)
2020 A(K,J)=A(K,J)/C
2030 NEXT J
2040 FOR S=1 TO NS
2050 IF S=K THEN GOTO 2100
2060 C=-A(S,K)
2070 FOR J=K TO (NS+1)
2080 A(S,J)=A(S,J)+(C*A(K,J))
2090 NEXT J
2100 NEXT S
2110 NEXT K
2120 XX(1)=A(1,NS+1)
2130 XX(2)=A(2,NS+1)
2140 XX(3)=A(3,NS+1)
2150 RETURN
```

Program for Shear Specimen 2

```
10 REM SHEAR SPECIMEN 2  NAME NEWFISH2
20 REM LPRINT "RESULT OF THE TEST DATA FOR SHEAR SPECIMEN-2"
30 REM LPRINT "0.5 TIMES THE BALANCE STEEL final2"
40 REM LPRINT "*********************************************"
50 LPRINT :LPRINT :LPRINT :LPRINT :LPRINT
60 DIM W(30),ST(30),S2(30),S4(30),S6(30),STRO(30),STR1(30),STR2(30),
   LEV(3,30),NAT(30),SA1(30),SA2(30),SA3(30),SAO(30),STR3(30),UN(30),
   COMP(3,30),AB(30),CM(3,30),AM(30),ACCU(3,30),ER(3,30),X(30),AS(30),Y(30),
   IC(30),YS(30),IS(30),ICCR(30)
70 DIM ISCR(30),ITCR(30),MCR(30),TI(30),IE(30),DEFC(30),ACCD(30),IT(30),
   DEFA(30),MA(30),A1(30),A2(30),A3(30),NX(30),YT(30),A(3,4),XX(3),
   TTX(30),TAX(30),TAT(30),XBAR(30),TAY(30)
80 N=19
90 FOR I=1 TO N
100 READ DEFA(I),W(I),ST(I),S2(I),S4(I),S6(I)
110 DATA 0,0,-12,-10,-16,+15
120 DATA 27,7880,-81.2,-53.15,-39.15,-18.35
130 DATA 60,16900,-181.4,-108.8,-56.55,-6.7
140 DATA 95,23820,-298.95,-152.4,-33.35,-44.45
150 DATA 138,31762.5,-423.35,-194,-28,-85.1
160 DATA 138,31762,-423.35,-194,-28,-65.3
170 DATA 165,39640,-534.6,-229.8,-20.3,-85.1
180 DATA 217,47350,-613.45,-268,-30.9,-63.85
190 DATA 260,53407,-731.1,-308.65,-36.7,-99.15
200 DATA 294,59159,-817.2,-343.5,-45.4,-106.4
210 DATA 336,59830,-793.5,-368.2,-37.7,7.75
220 DATA 354,63850,-840.9,-391.85,-40.6,7.75
230 DATA 374,67760,-896.1,-416.55,-45.4,4.85
240 DATA 395,71705,-947.4,-443.65,-50.3,4.35
250 DATA 418,75750,-994.8,-467.35,-49.3,12.1
260 DATA 430,79760,-1056.7,-497.85,-56.1,12.1
270 DATA 460,80365,-1085.8,-505.1,-50.3,11.15
280 DATA 472,83500,-1146.7,-532.65,-48.3,5.3
290 DATA 495,86700,-1138,-538.5,-39.6,2.45
300 SA1(I)=(S2(I)-S2(1))
310 SA2(I)=(S4(I)-S4(1))
320 SA3(I)=(S6(I)-S6(1))
330 SAO(I)=(ST(I)-ST(1))
340 NEXT I
350 REM LPRINT "-VE SIGN INDICATES TENSION":LPRINT :LPRINT
360 LPRINT "LOAD IN  NUTERAL AXIS  STRESS AT  STRESS AT  STRESS AT  STRESS AT"
370 LPRINT "  KIPS.   DEPTH IN       TOP          2 IN.        4 IN        6 IN."
380 FOR I=2 TO N
390 UN(I)=(((SAO(I)-SA1(I))/2)+((SA1(I)-SA2(I))/2)/2):PRINT "UN(";I;")=";UN(I)
400 NAT(I)=(4+(SA2(I)/UN(I)))
410 AA=1:BB=1:CC=1:DD=1
420 IF SGN(SA1(I))=1 THEN AA=0
430 IF SGN(SA2(I))=1 THEN BB=0
440 IF SGN(SA3(I))=1 THEN CC=0
450 IF SGN(SAO(I))=1 THEN DD=0
460 SA1(I)=ABS(SA1(I))
470 SA2(I)=ABS(SA2(I))
480 SA3(I)=ABS(SA3(I))
490 SAO(I)=ABS(SAO(I))
```

```
500 STRO(I)=(-4.06906E-07*(SAO(I)^3)+5.619357E-04*(SAO(I)^2)+6.060692*(SAO(I))+
    18.562801#)
510 STR1(I)=(-4.06906E-07*(SA1(I)^3)+5.619357E-04*(SA1(I)^2)+6.060692*(SA1(I))+
    18.562801#)
520 STR2(I)=(-4.06906E-07*(SA2(I)^3)+5.619357E-04*(SA2(I)^2)+6.060692*(SA2(I))+
    18.562801#)
530 STR3(I)=(-4.06906E-07*(SA3(I)^3)+5.619357E-04*(SA3(I)^2)+6.060692*(SA3(I))+
    18.562801#)
540 IF AA=0 THEN STR1(I)=-STR1(I)
550 IF BB=0 THEN STR2(I)=-STR2(I)
560 IF CC=0 THEN STR3(I)=-STR3(I)
570 LPRINT
580 W(I)=W(I)/1000
581 DVN=10
582 STRO(I)=(CINT(STRO(I)/DVN))*DVN
583 STR1(I)=(CINT(STR1(I)/DVN))*DVN
584 STR2(I)=(CINT(STR2(I)/DVN))*DVN
585 STR3(I)=(CINT(STR3(I)/DVN))*DVN
590 LPRINT USING " ###.#      #.##       #####     ######    ######    ######  ";
    W(I),NAT(I),STRO(I),STR1(I),STR2(I),STR3(I)
600 W(I)=W(I)*1000
610 NEXT I
620 LPRINT CHR$(12)
630 D=10.6875
640 FC=11400
650 B=.65:WB=8
660 FOR COUNTER=0 TO 2
670 IF COUNTER=0 THEN PRINT "ACI STRESS BLOCK RESULTS"
680 IF COUNTER=1 THEN PRINT "TRIANGULER STRESS BLOCK RESULTS"
690 IF COUNTER=2 THEN PRINT "PARABOLIC STRESS BLOCK RESULTS"
700 FOR I=1 TO N
710 AB(I)=(NAT(I)*B)
720 IF COUNTER=2 GOTO 830
730 IF COUNTER=1 GOTO 760
740 COMP(COUNTER,I)=(STRO(I)*AB(I)*WB)
750 IF COUNTER=0 GOTO 770
760 COMP(COUNTER,I)=(.5*STRO(I)*NAT(I)*WB)
770 PRINT "COMP(";COUNTER,I;")=";COMP(COUNTER,I)
780 IF COUNTER=1 GOTO 810
790 LEV(COUNTER,I)=(D-(AB(I)/2))
800 IF COUNTER=0 GOTO 1230
810 LEV(COUNTER,I)=(D-(NAT(I)/3))
820 IF COUNTER=0 OR 1 GOTO 1230
830 REM PARABALOIC CURVE FITTING
840 NP=3
850 IF NAT(I)>=4 THEN NP=4
860 X(1)=NAT(I)
870 X(2)=(NAT(I)-2)
880 IF NAT(I)<4 THEN X(3)=0:Y(3)=0:GOTO 900
890 X(3)=NAT(I)-4:X(4)=0:Y(3)=STR2(I):Y(4)=0:BEEP:BEEP:BEEP:BEEP:
900 Y(1)=STRO(I)
910 Y(2)=STR1(I)
920 SX=0:SX2=0:SX3=0:SX4=0:SY=0:SXY=0:SX2Y=0
930 FOR T=1 TO NP
940 SX=SX+X(T)
950 SY=SY+Y(T)
960 SX2=SX2+(X(T)^2)
970 SX3=SX3+(X(T)^3)
980 SX4=SX4+(X(T)^4)
990 SXY=SXY+X(T)*Y(T)
```

```
1000 SX2Y=SX2Y+(X(T)*X(T)*Y(T))
1010 NEXT T
1030 A(2,2)=SX2:A(2,3)=SX3
1040 A(3,2)=SX3:A(3,3)=SX4
1050 A(2,4)=SXY:A(3,4)=SX2Y
1060 GOSUB 1910
1070 A1(I)=XX(1)
1080 A2(I)=XX(2)
1090 A3(I)=XX(3)
1100 NX(I)=NAT(I):PRINT ;"A1(";I;")=";A1(I),A2(I),A3(I)
1110 TTX(I)=NX(I)/10
1120 FOR Z=1 TO 11
1130 TAX(Z)=NX(I)-((Z-1)*TTX(I))
1140 TAY(Z)=(A3(I)*(TAX(Z)^2))+(A2(I)*TAX(Z))+A3(I)
1150 NEXT Z
1160 FOR Z=1 TO 10
1170 TAT(I)=TAT(I)+(TTX(I)*((TAX(Z)*TAY(Z))+((TAY(Z)+TAY(Z+1))*
     (TAX(Z)+TAX(Z+1)))+(TAX(Z+1)*TAY(Z+1))))/6
1180 NEXT Z
1190 AS(I)=(A3(I)*((NX(I)^3)/3))+(A2(I)*((NX(I)^2)/2))+(A1(I)*NX(I))
1200 XBAR(I)=TAT(I)/AS(I):XBAR(I)=NX(I)-XBAR(I):PRINT "XBAR(";I;")=";XBAR(I)
1210 LEV(COUNTER,I)=(D-XBAR(I))
1220 COMP(COUNTER,I)=(AS(I)*8):PRINT "COMP(";COUNTER,I;")="COMP(COUNTER,I)
1230 PRINT "LEV(";COUNTER,I;")=";LEV(COUNTER,I)
1240 CM(COUNTER,I)=COMP(COUNTER,I)*LEV(COUNTER,I):L=84
1250 AM(I)=(W(I)*L)/6
1260 PRINT "CM(";COUNTER,I;")="CM(COUNTER,I),"AM(";I;")=";AM(I)
1270 ACCU(COUNTER,I)=(AM(I)/CM(COUNTER,I))
1280 ER(COUNTER,I)=((CM(COUNTER,I)-AM(I))/AM(I))*100
1290 PRINT "ACCU(";COUNTER,I;")=";ACCU(COUNTER,I):PRINT
     PRINT "ER(";COUNTER,I;")=";ER(COUNTER,I)
1300 AM(I)=AM(I)/12000:CM(COUNTER,I)=(CM(COUNTER,I)/12000)
1310 NEXT I
1320 NEXT COUNTER
1330 FOR I=1 TO N
1340 W(I)=W(I)/1000:DEFA(I)=DEFA(I)/1000
1350 NEXT I
1360 FOR COUNT=1 TO 3
1370 LPRINT :LPRINT :LPRINT :LPRINT
1380 IF COUNT=1 THEN LPRINT "LOAD NO.  LOAD IN  Mu (test)  Mu ([]calc)
     Mu(^calc)  Mu (test)      Mu(test)
1390 IF COUNT=2 THEN LPRINT "LOAD NO.  LOAD IN  Mu (test)  Mu (^ calc)
     Mu(~calc)  Mu (test)      Mu(test)
1400 IF COUNT=3 THEN LPRINT "LOAD NO.  LOAD IN  Mu (test)  Mu ([]calc)
     Mu(~calc)  Mu (test)      Mu(test)
1410 IF COUNT=1 THEN LPRINT "          KIPS      KIP-FT      KIP-FT
     KIP-FT     Mu ([]calc)    Mu(^calc)
1420 IF COUNT=2 THEN LPRINT "          KIPS      KIP-FT      KIP-FT
     KIP-FT     Mu (^ calc)    Mu(~calc)
1430 IF COUNT=3 THEN LPRINT "          KIPS      KIP-FT      KIP-FT
     KIP-FT     Mu ([]calc)    Mu(~calc)
1440 LPRINT
1450 FOR I=N TO N
1460IF COUNT=1 THEN LPRINT USING "   ##    ###.#  ###.# ###.#  ###.#
     ###.##  ###.##";I,W(I),AM(I),CM(0,I),CM(1,I),ACCU(0,I),ACCU(1,I)
1470 NEXT I
1480 FOR I=2 TO N
1490 IF COUNT=2 THEN LPRINT USING "   ##       ###.#     ###.#      ###.#
     ###.#  ###.##  ###.##";I,W(I),AM(I),CM(1,I),CM(2,I),ACCU(1,I),ACCU(2,I)
1500 IF I<N THEN GOTO 1520
```

```
1510 IF COUNT=3 THEN LPRINT USING "   ##   ####.#   ###.#   ###.#    ###.#
     ###.##  ###.##";I,W(I),AM(I),CM(0,I),CM(2,I),ACCU(0,I),ACCU(2,I)
1520 IF COUNT=2 THEN LPRINT
1530 NEXT I
1540 LPRINT CHR$(12)
1550 NEXT COUNT
1560 FOR COUNTER=0 TO 2
1570 FOR I=1 TO N
1580 CM(COUNTER,I)=(CM(COUNTER,I)*12000)
1590 NEXT I
1600 NEXT COUNTER
1610 LPRINT "LOAD NO.  LOAD IN  DEF ACTUAL    DEF CAL  DEF ACTUAL"
1620 LPRINT "            KIPS         IN.          IN.   DEF CAL"
1630 FOR I=2 TO N
1640 IF STRO(I)<6700 THEN EC=6216845!
1650 IF STRO(I)>6700 AND STRO(I)<9700 THEN EC=5467505!
1660 IF STRO(I)>9700 THEN EC=3828919!
1670 TD=12:WB=8:AS=1.8:ES=2.9E+07:A=96
1680 MA(I)=AM(I)
1690 YT(I)=(6-NAT(I))
1700 IC(I)=(WB*(TD^3))/12+(A*(YT(I)^2))
1710 YS(I)=(D-NAT(I))
1720 IS(I)=(((ES/EC)-1)*AS)*(YS(I)^2)
1730 IT(I)=IC(I)+IS(I):PRINT "ITCG=";IT(I)
1740 ICCR(I)=(WB*(NAT(I)^3)/12)+((WB*NAT(I))*((NAT(I)/2)^2))
1750 NS=(ES/EC)
1760 ISCR(I)=(NS*AS)*(YS(I)^2)
1770 ITCR(I)=ICCR(I)+ISCR(I):PRINT "ITCR=";ITCR(I)
1780 FCR=7.5*SQR(FC)
1790 MCR(I)=(FCR*IT(I))/(TD-NAT(I))
1800 TI(I)=((MCR(I)/(106.96*12000))^3)
1810 IE(I)=(TI(I)*IT(I))+((1-TI(I))*ITCR(I))
1820 IF (AM(I)*12000)<MCR(I) THEN IE(I)=IT(I)
1830 PRINT "TI=";IE(I)
1840 DEFC(I)=(23*W(I)*(84^3)*1000)/(1296*EC*IE(I))
1850 ACCD(I)=(DEFA(I)/DEFC(I))
1860 LPRINT
1870 LPRINT USING "    ##     ##.#     #.###         #.###       #.##";
     I,W(I),DEFA(I),DEFC(I),ACCD(I)
1880 NEXT I
1890 LPRINT CHR$(12)
1900 END
1910 NS=3
1920 FOR K=1 TO NS
1930 C=A(K,K)
1940 FOR J=K TO (NS+1)
1950 A(K,J)=A(K,J)/C
1960 NEXT J
1970 FOR S=1 TO NS
1980 IF S=K THEN GOTO 2030
1990 C=-A(S,K)
2000 FOR J=K TO (NS+1)
2010 A(S,J)=A(S,J)+(C*A(K,J))
2020 NEXT J
2030 NEXT S
2040 NEXT K
2050 XX(1)=A(1,NS+1)
2060 XX(2)=A(2,NS+1)
2070 XX(3)=A(3,NS+1)
2080 RETURN
```

Program for Under-reinforced Specimen

```
10 REM SHEAR SPECIMAN 1  NAME FINALUR
20 REM LPRINT "RESULT OF THE TEST DATA FOR UNDER-REINFORCED SPECIMEN-1"
30 REM LPRINT "0.5 TIMES THE BALANCE STEEL NAME FINALUR"
60DIM W(30),ST(30),S2(30),S4(30),S6(30),STR0(30),STR1(30),STR2(30),LEV(3,30),
   NAT(30),SA1(30),SA2(30),SA3(30),SA0(30),STR3(30),UN(30),COMP(3,30),AB(30),
   CM(3,30),AM(30),ACCU(3,30),ER(3,30),X(30),AS(30),Y(30),IC(30),YS(30),
   IS(30),ICCR(30)
70 DIM ISCR(30),ITCR(30),MCR(30),TI(30),IE(30),DEFC(30),ACCD(30),IT(30),
   DEFA(30),MA(30),A1(30),A2(30),A3(30),NX(30),YT(30),A(3,4),XX(3),TTX(30),
   TAX(30),TAT(30),XBAR(30),TAY(30)
80 N=21
90 FOR I=1 TO N
100 READ DEFA(I),W(I),ST(I),S2(I),S4(I),S6(I)
110 DATA 0,0,4,1,-3,-2
120 DATA 48,11810,-117.5,-59.95,-23.2,1.9
130 DATA 116,25000,-327.5,-124.3,-30.95,24.1
140 DATA 170,35670,-477.05,-167.85,-42.55,1.3
150 DATA 230,47820,-650.75,-217.2,47.45,9.15
160 DATA 294,59520,-810.45,-268,140.3,11.1
170 DATA 324,67570,-919.8,-306.75,161.61,7.7
180 DATA 378,75400,-1066.4,-349.8,254.5,11.15
190 DATA 424,83460,-1202.4,-397.7,315.4,12.55
200 DATA 450,87450,-1280.25,-418.5,363.8,15
210 DATA 478,89200,-1473.3,-512.4,665.8,15.0
220 DATA 582,85100,-1502.35,-371.05,561.2,7.25
230 DATA 640,85100,-1694.95,-338.2,450.9,34.85
240 DATA 685,87760,-1773.35,-325.1,387,15.45
250 DATA 755,88660,-1963.5,-254.95,492.5,51.15
260 DATA 780,91000,-2210.3,-171.25,564.1,55.15
270 DATA 780,91510,-2121.85,-138.35,530.6,24.65
280 DATA 780,91680,-2413,-140.75,502.2,15.5
290 DATA 780,90310,-2213.25,-234.15,331.9,21.3
300 DATA 780,90800,-2096.05,-310.1,160.6,41.55
310 DATA 780,87450,-1280.25,-418.5,150.35,15
320 SA1(I)=(S2(I)-S2(1))
330 SA2(I)=(S4(I)-S4(1))
340 SA3(I)=(S6(I)-S6(1))
350 SA0(I)=(ST(I)-ST(1))
360 NEXT I
370 REM LPRINT "-VE SIGN INDICATES TENSION":LPRINT :LPRINT
380 LPRINT "LOAD IN  NEUTRAL AXIS STRESS AT  STRESS AT  STRESS AT STRESS AT"
390 LPRINT "  KIPS.   DEPTH IN     TOP         2 IN.       4 IN        6 IN."
400 FOR I=2 TO N
410 UN(I)=(SA0(I)-SA1(I))/2 :PRINT "UN(";I;")=";UN(I):PRINT
420 IF SGN(SA2(I))=-1 THEN GOTO 440
430 NAT(I)=(2+(SA1(I)/UN(I))):PRINT"NAT(";I;")=";NAT(I):GOTO 450
440 NAT(I)=(4+(SA2(I)/UN(I)))
450 AA=1:BB=1:CC=1:DD=1
460 IF SGN(SA1(I))=1 THEN AA=0
470 IF SGN(SA2(I))=1 THEN BB=0
480 IF SGN(SA3(I))=1 THEN CC=0
490 IF SGN(SA0(I))=1 THEN DD=0
500 SA1(I)=ABS(SA1(I))
510 SA2(I)=ABS(SA2(I))
520 SA3(I)=ABS(SA3(I))
530 SA0(I)=ABS(SA0(I))
```

```
540 STRO(I)=(-4.06906E-07*(SAO(I)^3)+5.619357E-04*(SAO(I)^2)+6.060692*(SAO(I))+
    18.562801#)
550 STR1(I)=(-4.06906E-07*(SA1(I)^3)+5.619357E-04*(SA1(I)^2)+6.060692*(SA1(I))+
    18.562801#)
560 STR2(I)=(-4.06906E-07*(SA2(I)^3)+5.619357E-04*(SA2(I)^2)+6.060692*(SA2(I))+
    18.562801#)
570 STR3(I)=(-4.06906E-07*(SA3(I)^3)+5.619357E-04*(SA3(I)^2)+6.060692*(SA3(I))+
    18.562801#)
580 IF AA=0 THEN STR1(I)=-STR1(I)
590 IF BB=0 THEN STR2(I)=-STR2(I)
600 IF CC=0 THEN STR3(I)=-STR3(I)
610 LPRINT
620 W(I)=W(I)/1000
621 DVN=10
622 IF STRO(I)>=10000 THEN DVN=100
623 STRO(I)=(CINT(STRO(I)/DVN))*DVN:DVN=10
624 STR1(I)=(CINT(STR1(I)/DVN))*DVN
625 STR2(I)=(CINT(STR2(I)/DVN))*DVN
626 STR3(I)=(CINT(STR3(I)/DVN))*DVN
630 LPRINT USING " ###.#    #.##    #####   ######  ######  ######   ";
    W(I),NAT(I), STRO(I),STR1(I),STR2(I),STR3(I)
640 W(I)=W(I)*1000
650 NEXT I
660 LPRINT CHR$(12)
670 D=10.6875
680 FC=11700
690 B=.65:WB=8
700 FOR COUNTER=0 TO 2
710 IF COUNTER=0 THEN PRINT "ACI STRESS BLOCK RESULTS"
720 IF COUNTER=1 THEN PRINT "TRIANGULER STRESS BLOCK RESULTS"
730 IF COUNTER=2 THEN PRINT "PARABOLIC STRESS BLOCK RESULTS"
740 FOR I=1 TO N
750 AB(I)=(NAT(I)*B)
760 IF COUNTER=2 GOTO 870
770 IF COUNTER=1 GOTO 800
780 COMP(COUNTER,I)=(.85*FC*AB(I)*WB)
790 IF COUNTER=0 GOTO 810
800 COMP(COUNTER,I)=(.5*STRO(I)*NAT(I)*WB)
810 PRINT "COMP(";COUNTER,I;")=";COMP(COUNTER,I)
820 IF COUNTER=1 GOTO 850
830 LEV(COUNTER,I)=(D-(AB(I)/2))
840 IF COUNTER=0 GOTO 1270
850 LEV(COUNTER,I)=(D-(NAT(I)/3))
860 IF COUNTER=0 OR 1 GOTO 1270
870 REM PARABALOIC CURVE FITTING
880 NP=3
890 IF NAT(I)>=4 THEN NP=4
900 X(1)=NAT(I)
910 X(2)=(NAT(I)-2)
920 IF NAT(I)<4 THEN X(3)=0:Y(3)=0:GOTO 940
930 X(3)=NAT(I)-4:X(4)=0:Y(3)=STR2(I):Y(4)=0:BEEP:BEEP:BEEP:BEEP:
940 Y(1)=STRO(I)
950 Y(2)=STR1(I)
960 SX=0:SX2=0:SX3=0:SX4=0:SY=0:SXY=0:SX2Y=0
970 FOR T=1 TO NP
980 SX=SX+X(T)
990 SY=SY+Y(T)
1000 SX2=SX2+(X(T)^2)
1010 SX3=SX3+(X(T)^3)
1020 SX4=SX4+(X(T)^4)
```

```
1030 SXY=SXY+X(T)*Y(T)
1040 SX2Y=SX2Y+(X(T)*X(T)*Y(T))
1050 NEXT T
1070 A(2,2)=SX2:A(2,3)=SX3
1080 A(3,2)=SX3:A(3,3)=SX4
1090 A(2,4)=SXY:A(3,4)=SX2Y
1100 GOSUB 1970
1110 A1(I)=XX(1)
1120 A2(I)=XX(2)
1130 A3(I)=XX(3)
1140 NX(I)=NAT(I):PRINT ;"A1(";I;")=";A1(I),A2(I),A3(I)
1150 TTX(I)=NX(I)/10
1160 FOR Z=1 TO 11
1170 TAX(Z)=NX(I)-((Z-1)*TTX(I)):REM LPRINT "TAX(";Z;")=";TAX(Z)
1180 TAY(Z)=(A3(I)*(TAX(Z)^2))+(A2(I)*TAX(Z))+A1(I):REM LPRINT
     "TAY(";Z;")=";TAY(Z)
1190 NEXT Z
1200 FOR Z=1 TO 10
1210 TAT(I)=TAT(I)+(TTX(I)*((TAX(Z)*TAY(Z))+((TAY(Z)+TAY(Z+1))*
     (TAX(Z)+TAX(Z+1)))+(TAX(Z+1)*TAY(Z+1))))/6
1220 NEXT Z
1230 AS(I)=(A3(I)*((NX(I)^3)/3))+(A2(I)*((NX(I)^2)/2))+(A1(I)*NX(I)):
1240 XBAR(I)=TAT(I)/AS(I):XBAR(I)=NX(I)-XBAR(I)
1250 LEV(COUNTER,I)=(D-XBAR(I))
1260 COMP(COUNTER,I)=(AS(I)*8):PRINT "COMP(";COUNTER,I;")="COMP(COUNTER,I)
1270 PRINT "LEV(";COUNTER,I;")=";LEV(COUNTER,I)
1280 CM(COUNTER,I)=COMP(COUNTER,I)*LEV(COUNTER,I):L=84
1290 AM(I)=(W(I)*L)/6
1300 PRINT "CM(";COUNTER,I;")="CM(COUNTER,I),"AM(";I;")=";AM(I)
1310 ACCU(COUNTER,I)=(AM(I)/CM(COUNTER,I))
1320 ER(COUNTER,I)=((CM(COUNTER,I)-AM(I))/AM(I))*100
1330 PRINT "ACCU(";COUNTER,I;")=";ACCU(COUNTER,I):PRINT
     PRINT "ER(";COUNTER,I;")=";ER(COUNTER,I)
1340 AM(I)=AM(I)/12000:CM(COUNTER,I)=(CM(COUNTER,I)/12000)
1350 NEXT I
1360 NEXT COUNTER
1370 FOR I=1 TO N
1380 W(I)=W(I)/1000:DEFA(I)=DEFA(I)/1000
1390 NEXT I
1400 FOR COUNT=1 TO 3
1410 LPRINT :LPRINT :LPRINT:LPRINT :LPRINT :LPRINT :LPRINT :
1420 IF COUNT=1 THEN LPRINT "LOAD NO.  LOAD IN  Mu (test)  Mu ([]calc)
     Mu(^calc)  Mu (test)      Mu(test)
1430 IF COUNT=2 THEN LPRINT "LOAD NO.  LOAD IN  Mu (test)  Mu (^ calc)
     Mu( calc)  Mu (test)      Mu(test)
1440 IF COUNT=3 THEN LPRINT "LOAD NO.  LOAD IN  Mu (test)  Mu ([]calc)
     Mu( calc)  Mu (test)      Mu(test)
1450 IF COUNT=1 LPRINT "   KIPS  KIP-FT  KIP-FT KIP-FT  Mu ([]calc) Mu(^calc)
1460 IF COUNT=2 LPRINT "   KIPS  KIP-FT  KIP-FT KIP-FT  Mu (^ calc) Mu(_calc)
1470 IF COUNT=3 LPRINT "   KIPS  KIP-FT  KIP-FT KIP-FT  Mu ([]calc) Mu( calc)
1480 LPRINT
1490 FOR I=18 TO 18
1500 IF COUNT=1 THEN LPRINT USING "  ##     ###.#      ###.#      ###.##
     ###.#  ###.##  ###.##";I,W(I),AM(I),CM(0,I),CM(1,I),ACCU(0,I),ACCU(1,I)
1510 NEXT I
1520 FOR I= 2 TO N
1530 IF COUNT=2 THEN LPRINT USING "  ##   ###.#   ###.#  ###.#   ####.#
     ###.##   ###.##";I,W(I),AM(I),CM(1,I),CM(2,I),ACCU(1,I),ACCU(2,I)
1540 IF COUNT=2 THEN LPRINT
1550 NEXT I
```

```
1560 FOR I=18 TO 18
1570 IF COUNT=3 THEN LPRINT USING "   ##    ###.#    ###.#  ###.#    ####.#
     ###.##        ###.##";I,W(I),AM(I),CM(0,I),CM(2,I),ACCU(0,I),ACCU(2,I)
1580 NEXT I
1590 LPRINT CHR$(12)
1600 NEXT COUNT
1610 FOR COUNTER=0 TO 2
1620 FOR I=1 TO N
1630 CM(COUNTER,I)=(CM(COUNTER,I)*12000)
1640 NEXT I
1650 NEXT COUNTER
1660 LPRINT :LPRINT :LPRINT :LPRINT :LPRINT :LPRINT :LPRINT
1670 LPRINT "LOAD NO.  LOAD IN  DEF ACTUAL    DEF CAL  DEF ACTUAL"
1680 LPRINT "            KIPS         IN.          IN.   DEF CAL"
1690 FOR I=2 TO 16
1700 IF STRO(I)<6700 THEN EC=6216845!
1710 IF STRO(I)>6700 AND STRO(I)<9700 THEN EC=5467505!
1720 IF STRO(I)>9700 THEN EC=3828919!
1730 TD=12:WB=8:AS=1.8:ES=2.9E+07:A=96
1740 MA(I)=AM(I)
1750 YT(I)=(6-NAT(I))
1760 IC(I)=(WB*(TD^3))/12+(A*(YT(I)^2))
1770 YS(I)=(D-NAT(I))
1780 IS(I)=(((ES/EC)-1)*AS)*(YS(I)^2)
1790 IT(I)=IC(I)+IS(I):PRINT "ITCG=";IT(I)
1800 ICCR(I)=(WB*(NAT(I)^3)/12)+((WB*NAT(I))*((NAT(I)/2)^2))
1810 NS=(ES/EC)
1820 ISCR(I)=(NS*AS)*(YS(I)^2)
1830 ITCR(I)=ICCR(I)+ISCR(I):PRINT "ITCR=";ITCR(I)
1840 FCR=7.5*SQR(FC)
1850 MCR(I)=(FCR*IT(I))/(TD-NAT(I)):PRINT "&&&&&&&&&";MCR(I)
1860 TI(I)=((MCR(I)/(106.96*12000))^3)
1870 IE(I)=(TI(I)*IT(I))+((1-TI(I))*ITCR(I))
1880 IF (AM(I)*12000)<MCR(I) THEN IE(I)=IT(I)
1890 PRINT "TI=";IE(I):PRINT "EC="EC
1900 DEFC(I)=(23*W(I)*(84^3)*1000)/(1296*EC*IE(I))
1910 ACCD(I)=(DEFA(I)/DEFC(I))
1920 LPRINT
1930 LPRINT USING "## ##.# #.### #.### #.##";I,W(I),DEFA(I),DEFC(I),ACCD(I)
1940 NEXT I
1950 LPRINT CHR$(12)
1960 END
1970 NS=3
1980 FOR K=1 TO NS
1990 C=A(K,K)
2000 FOR J=K TO (NS+1)
2010 A(K,J)=A(K,J)/C
2020 NEXT J
2030 FOR S=1 TO NS
2040 IF S=K THEN GOTO 2090
2050 C=-A(S,K)
2060 FOR J=K TO (NS+1)
2070 A(S,J)=A(S,J)+(C*A(K,J))
2080 NEXT J
2090 NEXT S
2100 NEXT K
2110 XX(1)=A(1,NS+1)
2120 XX(2)=A(2,NS+1)
2130 XX(3)=A(3,NS+1)
2140 RETURN
```

Program for Over-reinforced Specimen

```
10 REM OVR-REIN SPECIMAN 1  NAME FINAL2
20 REM LPRINT "RESULT OF THE TEST DATA FOR OVER-REINFORCED PCIMEN-1"
30 REM LPRINT "1.5 TIMES THE BALANCE STEEL final2"
50 DIM W(30),ST(30),S2(30),S4(30),S6(30),STRO(30),STR1(30),STR2(30),
   LEV(3,30),NAT(30),SA1(30),SA2(30),SA3(30),SAO(30),STR3(30),UN(30),
   COMP(3,30),AB(30),CM(3,30),AM(30),ACCU(3,30),ER(3,30),X(30),AS(30),Y(30),
   IC(30),YS(30),IS(30),ICCR(30)
60 DIM ISCR(30),ITCR(30),MCR(30),TI(30),IE(30),DEFC(30),ACCD(30),IT(30),
   DEFA(30),MA(30),A1(30),A2(30),A3(30),NX(30),YT(30),A(3,4),XX(3),TTX(30),
   TAX(30),TAT(30),XBAR(30),TAY(30)
70 N=27
80 FOR I=1 TO N
90 READ DEFA(I),W(I),ST(I),S2(I),S4(I),S6(I)
100 DATA 0,0,-0.9,-1.0,-0.4,0
110 DATA 132,10800,-112.73,-76.93,-39.18,5.79
120 DATA 162,19770,-202.23,-135.47,-60.47,14.02
130 DATA 200,30000,-319.825,-208.05,-83.69,35.315
140 DATA 232,40000,-442.73,-284.51,-107.89,126.765
150 DATA 268,50000,-563.21,-359.025,-128.7,274.38
160 DATA 306,60000,-658.63,-435.47,-149.025,412.25
170 DATA 342,70000,-810.47,-513.375,-173.7,530.315
180 DATA 378,80000,-943.545,-596.125,-198.86,382.89
190 DATA 418,90000,-1078.05,-680.79,-229.35,731.125
200 DATA 440,99500,-1213.535,-769.35,-259.35,820.635
210 DATA 461,105620,-1305.465,-828.465,-279.665,865.64
220 DATA 486,112000,-1391.6,-884.99,-303.86,902.9
230 DATA 504,118000,-1480.63,-994.02,-324.67,923.38
240 DATA 532,124000,-1571.08,-1004.025,-345.75,964.34
250 DATA 558,136000,-1668.34,-1067.89,-370.63,986.6
260 DATA 583,142000,-1758.8,-1126.92,-392.4,1016.6
270 DATA 583,148000,-1857.53,-1192.24,-418.54,1038.37
280 DATA 583,154000,-1972.665,-1334.99,-474.66,1073.216
290 DATA 583,160000,-2073.33,-1334.99,-474.66,1073.216
300 DATA 583,166000,-2174.41,-1398.86,-500.315,1101.275
310 DATA 583,170000,-2291.5,-1474.35,-531.28,1119.185
320 DATA 583,172000,-2382.5,-1535.31,-554.51,1131.75
330 DATA 583,174000,-2440.58,-1574.99,-573.86,1132.24
340 DATA 583,176000,-2497.67,-1613.669,-590.79,1132.25
350 DATA 583,178000,-2561.98,-1681.44,-609.665,1130.3
360 DATA 583,180000,-2604.415,-1695.94,-625.635,1145.79
370 SA1(I)=(S2(I)-S2(1))
380 SA2(I)=(S4(I)-S4(1))
390 SA3(I)=(S6(I)-S6(1))
400 SAO(I)=(ST(I)-ST(1))
410 NEXT I
420 REM LPRINT "-VE SIGN INDICATES TENSION":LPRINT :LPRINT
430 LPRINT "LOAD IN NEUTRAL AXIS STRESS AT STRESS AT STRESS AT STRESS AT"
440 LPRINT "KIPS. DEPTH IN    TOP     2 IN.     4 IN      6 IN."
460 FOR I=2 TO N
470 UN(I)=(((SAO(I)-SA1(I))/2+(SA1(I)-SA2(I))/2))/2:PRINT "UN(";I;")=";UN(I)
480 NAT(I)=(4+(SA2(I)/UN(I))):PRINT"NAT(";I;")=";NAT(I):PRINT
510 AA=1:BB=1:CC=1:DD=1
520 IF SGN(SA1(I))=1 THEN AA=0
530 IF SGN(SA2(I))=1 THEN BB=0
540 IF SGN(SA3(I))=1 THEN CC=0
550 IF SGN(SAO(I))=1 THEN DD=0
```

```
560 SA1(I)=ABS(SA1(I))
570 SA2(I)=ABS(SA2(I))
580 SA3(I)=ABS(SA3(I))
590 SA0(I)=ABS(SA0(I))
600 STR0(I)=(-4.06906E-07*(SA0(I)^3)+5.619357E-04*(SA0(I)^2)+6.060692*(SA0(I))+
    18.562801#)
610 STR1(I)=(-4.06906E-07*(SA1(I)^3)+5.619357E-04*(SA1(I)^2)+6.060692*(SA1(I))+
    18.562801#)
620 STR2(I)=(-4.06906E-07*(SA2(I)^3)+5.619357E-04*(SA2(I)^2)+6.060692*(SA2(I))+
      18.562801#)
630 STR3(I)=(-4.06906E-07*(SA3(I)^3)+5.619357E-04*(SA3(I)^2)+6.060692*(SA3(I))+
      18.562801#)
640 IF AA=0 THEN STR1(I)=-STR1(I)
650 IF BB=0 THEN STR2(I)=-STR2(I)
660 IF CC=0 THEN STR3(I)=-STR3(I)
670 NEXT I
680 FOR I=2 TO N
690 W(I)=W(I)/1000
691 DVN=10
692 IF STR0(I)>=10000 THEN DVN=100
693 STR0(I)=(CINT(STR0(I)/DVN))*DVN:DVN=10
694 STR1(I)=(CINT(STR1(I)/DVN))*DVN
695 STR2(I)=(CINT(STR2(I)/DVN))*DVN
696 STR3(I)=(CINT(STR3(I)/DVN))*DVN
700 LPRINT USING "###.# #.##  ##### ###### ###### ######   ";W(I),
    NAT(I),STR0(I),STR1(I),STR2(I),STR3(I)
720 W(I)=W(I)*1000
730 NEXT I
740 LPRINT CHR$(12)
750 D=9.3125
760 FC=12100
770 B=.65:WB=8
780 FOR COUNTER=0 TO 2
790 IF COUNTER=0 THEN PRINT "ACI STRESS BLOCK RESULTS"
800 IF COUNTER=1 THEN PRINT "TRIANGULER STRESS BLOCK RESULTS"
810 IF COUNTER=2 THEN PRINT "PARABOLIC STRESS BLOCK RESULTS"
820 FOR I=1 TO N
830 AB(I)=(NAT(I)*B)
840 IF COUNTER=2 GOTO 950
850 IF COUNTER=1 GOTO 880
860 COMP(COUNTER,I)=(.85*FC*AB(I)*WB)
870 IF COUNTER=0 GOTO 890
880 COMP(COUNTER,I)=(.5*STR0(I)*NAT(I)*WB)
890 PRINT "COMP(";COUNTER,I;")=";COMP(COUNTER,I)
900 IF COUNTER=1 GOTO 930
910 LEV(COUNTER,I)=(D-(AB(I)/2))
920 IF COUNTER=0 GOTO 1350
930 LEV(COUNTER,I)=(D-(NAT(I)/3))
940 IF COUNTER=0 OR 1 GOTO 1350
950 REM PARABALOIC CURVE FITTING
960 NP=3
970 IF NAT(I)>=4 THEN NP=4
980 X(1)=NAT(I)
990 X(2)=(NAT(I)-2)
1000 IF NAT(I)<4 THEN X(3)=0:Y(3)=0:GOTO 1020
1010 X(3)=NAT(I)-4:X(4)=0:Y(3)=STR2(I):Y(4)=0:BEEP:BEEP:BEEP:BEEP:
1020 Y(1)=STR0(I)
1030 Y(2)=STR1(I)
1040 SX=0:SX2=0:SX3=0:SX4=0:SY=0:SXY=0:SX2Y=0
1050 FOR T=1 TO NP
```

```
1060 SX=SX+X(T)
1070 SY=SY+Y(T)
1080 SX2=SX2+(X(T)^2)
1090 SX3=SX3+(X(T)^3)
1100 SX4=SX4+(X(T)^4)
1110 SXY=SXY+X(T)*Y(T)
1120 SX2Y=SX2Y+(X(T)*X(T)*Y(T))
1130 NEXT T
1155 A(2,2)=SX2:A(2,3)=SX3
1165 A(3,2)=SX3:A(3,3)=SX4
1170 A(2,4)=SXY:A(3,4)=SX2Y
1180 GOSUB 2040
1190 A1(I)=XX(1)
1200 A2(I)=XX(2)
1210 A3(I)=XX(3)
1220 NX(I)=NAT(I):PRINT ;"A1(";I;")=";A1(I),A2(I),A3(I)
1230 TTX(I)=NX(I)/10
1240 FOR Z=1 TO 11
1250 TAX(Z)=NX(I)-((Z-1)*TTX(I))
1260 TAY(Z)=(A3(I)*(TAX(Z)^2))+(A2(I)*TAX(Z))+A1(I)
1270 NEXT Z
1280 FOR Z=1 TO 10
1290 TAT(I)=TAT(I)+(TTX(I)*((TAX(Z)*TAY(Z))+((TAY(Z)+TAY(Z+1))*
     (TAX(Z)+TAX(Z+1)))+(TAX(Z+1)*TAY(Z+1))))/6
1300 NEXT Z
1310 AS(I)=(A3(I)*((NX(I)^3)/3))+(A2(I)*((NX(I)^2)/2))+(A1(I)*NX(I))
1320 XBAR(I)=TAT(I)/AS(I):XBAR(I)=NX(I)-XBAR(I)
1330 LEV(COUNTER,I)=(D-XBAR(I))
1340 COMP(COUNTER,I)=(AS(I)*8):PRINT "COMP(";COUNTER,I;")="COMP(COUNTER,I)
1350 PRINT "LEV(";COUNTER,I;")=";LEV(COUNTER,I)
1360 CM(COUNTER,I)=COMP(COUNTER,I)*LEV(COUNTER,I):L=84
1370 AM(I)=(W(I)*L)/6
1380 PRINT "CM(";COUNTER,I;")="CM(COUNTER,I),"AM(";I;")=";AM(I)
1390 ACCU(COUNTER,I)=(AM(I)/CM(COUNTER,I))
1400 ER(COUNTER,I)=((CM(COUNTER,I)-AM(I))/AM(I))*100
1410 PRINT "ACCU(";COUNTER,I;")=";ACCU(COUNTER,I):PRINT :
     PRINT "ER(";COUNTER,I;")=";ER(COUNTER,I)
1420 AM(I)=AM(I)/12000:CM(COUNTER,I)=(CM(COUNTER,I)/12000)
1430 NEXT I
1440 NEXT COUNTER
1450 FOR I=1 TO N
1460 W(I)=W(I)/1000:DEFA(I)=DEFA(I)/1000
1470 NEXT I
1480 FOR COUNT=1 TO 3
1500 IF COUNT=1 THEN LPRINT "LOAD NO.  LOAD IN  Mu (test)  Mu ([]calc)
     Mu(^calc)  Mu (test)      Mu(test)
1510 IF COUNT=2 THEN LPRINT "LOAD NO.  LOAD IN  Mu (test)  Mu (^ calc)
     Mu(¯calc)  Mu (test)      Mu(test)
1520 IF COUNT=3 THEN LPRINT "LOAD NO.  LOAD IN  Mu (test)  Mu ([]calc)
     Mu(¯calc)  Mu (test)      Mu(test)
1530 IF COUNT=1 THEN LPRINT "           KIPS      KIP-FT      KIP-FT
     KIP-FT      Mu ([]calc)    Mu(^calc)
1540 IF COUNT=2 THEN LPRINT "           KIPS      KIP-FT      KIP-FT
     KIP-FT      Mu (^ calc)    Mu(¯calc)
1550 IF COUNT=3 THEN LPRINT "           KIPS      KIP-FT      KIP-FT
     KIP-FT      Mu ([]calc)    Mu(¯calc)"
1570 FOR I=N TO N
1580 IF COUNT=1 THEN LPRINT USING " ## ###.#  ###  ### ### #.##  #.##   ";
     I,W(I),AM(I),CM(0,I),CM(1,I),ACCU(0,I),ACCU(1,I)
1590 NEXT I
```

```
1600 FOR I=2 TO N
1610 IF COUNT=2 THEN LPRINT USING " ##  ###.# ###  ### ### #.##   #.##   ";
     I,W(I),AM(I),CM(1,I),CM(2,I),ACCU(1,I),ACCU(2,I)
1620 IF I<N THEN GOTO 1640
1630 IF COUNT=3 THEN LPRINT USING " ##  ###.#  ###  ### ### #.##  #.##   ";
     I,W(I),AM(I),CM(0,I),CM(2,I),ACCU(0,I),ACCU(2,I)
1640 IF COUNT=2 THEN LPRINT
1650 NEXT I
1670 NEXT COUNT
1680 FOR COUNTER=0 TO 2
1690 FOR I=1 TO N
1700 CM(COUNTER,I)=(CM(COUNTER,I)*12000)
1710 NEXT I
1720 NEXT COUNTER
1740 LPRINT "LOAD NO.  LOAD IN  DEF ACTUAL    DEF CAL  DEF ACTUAL"
1750 LPRINT "            KIPS          IN.          IN.  DEF CAL"
1760 FOR I=2 TO 17
1770 IF STRO(I)<6700 THEN EC=6216845!
1780 IF STRO(I)>6700 AND STRO(I)<9700 THEN EC=5467505!
1790 IF STRO(I)>9700 THEN EC=3828919!
1800 TD=12:WB=8:AS=1.8:ES=2.9E+07:A=96
1810 MA(I)=AM(I)
1820 YT(I)=(6-NAT(I))
1830 IC(I)=(WB*(TD^3))/12+(A*(YT(I)^2))
1840 YS(I)=(D-NAT(I))
1850 IS(I)=(((ES/EC)-1)*AS)*(YS(I)^2)
1860 IT(I)=IC(I)+IS(I):PRINT "ITCG=";IT(I)
1870 ICCR(I)=(WB*(NAT(I)^3)/12)+((WB*NAT(I))*((NAT(I)/2)^2))
1880 NS=(ES/EC)
1890 ISCR(I)=(NS*AS)*(YS(I)^2)
1900 ITCR(I)=ICCR(I)+ISCR(I):PRINT "ITCR=";ITCR(I)
1910 FCR=7.5*SQR(FC)
1920 MCR(I)=(FCR*IT(I))/(TD-NAT(I))
1930 TI(I)=((MCR(I)/(106.96*12000))^3)
1940 IE(I)=(TI(I)*IT(I))+((1-TI(I))*ITCR(I))
1950 IF AM(I)*12000<MCR(I) THEN IE(I)=IT(I)
1960 PRINT "TI=";IE(I)
1970 DEFC(I)=(23*W(I)*(84^3)*1000)/(1296*EC*IE(I))
1980 ACCD(I)=(DEFA(I)/DEFC(I))
2000 LPRINT USING "## ### #.### #.###  #.##";I,W(I),DEFA(I),DEFC(I),ACCD(I)
2010 NEXT I
2030 END
2040 NS=3
2050 FOR K=1 TO NS
2060 C=A(K,K)
2070 FOR J=K TO (NS+1)
2080 A(K,J)=A(K,J)/C
2090 NEXT J
2100 FOR S=1 TO NS
2110 IF S=K THEN GOTO 2160
2120 C=-A(S,K)
2130 FOR J=K TO (NS+1)
2140 A(S,J)=A(S,J)+(C*A(K,J))
2150 NEXT J
2160 NEXT S
2170 NEXT K
2180 XX(1)=A(1,NS+1)
2190 XX(2)=A(2,NS+1)
2200 XX(3)=A(3,NS+1)
2210 RETURN
```

APPENDIX IV

TABLES AND FIGURES

Table 3.1: Tensile Test Results for Steel Reinforcing Bars.

Bar No.	Area sq. in.	Yield Load in lbs.	Ultimate Load in lbs.	Yield Stress in psi	Ultimate Stress in psi
3	0.11	6625	7600	60000	69100
3	0.11	7400	10300	67000	93600
4	0.20	12400	16450	62000	82200
4	0.20	12150	16050	61000	80200
7	0.60	38800	63600	64700	106000
7	0.60	38000	63600	64700	106000
9	1.00	63800	97300	63800	97300

Mean Yield Stress of #3 bars =63500 psi

Mean Yield Stress of #4 bars =61500 psi

Mean Yield Stress of #7 bars =64700 psi

Mean Yield Stress of #9 bars =63800 psi

1 lb. = 4.45 N

1 psi = 6.89 kPa

Table 4.1: Compressive Strength (3 days) Test Results of 3 in. X 6 in. Cylinders Made By Regular Mixing Technique

Cylinder No.	Crushing Load lbs.	Crushing Strength psi
1	44400	6300
2	50500	7100
3	51500	7300
4	43000	6100

Average Cylinder Compressive Strength f'_c = 6700 psi

Population Standard Deviation. σ = 590 psi

Coefficient of Variation V = 8.8%

1 lb. = 4.45 N

1 psi = 6.89 kPa

Table 4.2: Compressive Strength (3 days) Test Results of 3 in. X 6 in. Cylinders Made by Cement Slurry Method

Cylinder No.	Crushing Load lbs.	Crushing Strength psi
1	48500	6900
2	45000	6400
3	48000	6800
4	51500	7300

Average Cylinder Compressive Strength f'_c = 6800 psi

Population Standard Deviation. σ = 370 psi

Coefficient of Variation V = 5.0%

1 lb. = 4.45 N

1 psi = 6.89 kPa

Table 5.1: Compressive Strength (28 days) Test Results of 3 in. X 6 in. Cylinders Made from the Mix Proportions taken from Ref. 15

Cylinder No.	Crushing Load lbs.	crushing Strength psi
1	60700	8600
2	63500	9000
3	65700	9300
4	63700	9000
5	60000	8500
6	60100	8500

Average Cylinder Compressive Strength f'_c = 8800 psi

Population Standard Deviation. σ = 330 psi

Coefficient of Variation V = 4.0%

1 lb. = 4.45 N

1 psi = 6.89 kPa

Table 5.2: Compressive Strength Test Results of 3 in. X 6 in. Cylinders for Beam 1 (SS1B)

Cylinder No.	Crushing Load lbs.	Crushing Strength psi
SS1A-1	74500	10500
SS1A-2	81000	11500
SS1B-1	62000	8770
SS1B-2	64500	9120
SSIB-3	56000	7920
SS1B-4	68000	9620
SS1B-5	64000	9050

Average Cylinder Compressive Strength f'_c = 9500 psi

Population Standard Deviation. σ = 1180 psi

Coefficient of Variation V = 12.5%

1 lb. = 4.45 N

1 psi = 6.89 kpa.

Table 5.3: Compressive Strength Test Results of 3 in. X 6 in. Cylinders for Beam 2 (SS2B)

Cylinder No.	Crushing Load lbs.	Crushing Strength psi
SS2A-1	75000	10600
SS2A-2	83200	11700
SS2B-1	84200	11900
SS2B-2	75000	10600
SS2B-3	85000	12000

Average Cylinder Compressive Strength f'_c = 11400

Population Standard Deviation. σ = 700 psi

Coefficient of Variation V = 6.2%

1 lb. = 4.45 N

1 psi = 6.89 kPa

Table 5.4 : Compressive Strength Test Results of 3 in. X 6 in. Cylinders for Beam 3 (UR1)

Cylinder No.	Crushing Load lbs.	Crushing Strength psi
UR1-1	84800	12000
UR1-2	74000	10500
UR1-3	82200	11600
UR1-4	85800	12100
UR1-5	85400	12000
UR1-6	86400	12200

Average Cylinder Compressive Strength f'_c = 11700 psi

Population Standard Deviation. σ = 640 psi

Coefficient of Variation V = 5.4%

1 lb. = 4.45 N

1 psi = 6.89 kPa

Table 5.5 : Compressive Strength Test Results of 3 in. X 6 in. Cylindera for Beam 4 (OR1)

Cylinder No.	Crushing Load lbs.	crushing Strength psi
OR1-1	83500	11800
OR1-2	84000	11900
OR1-3	83000	11700
OR1-4	91000	12900
OR1-5	76000	10800
OR1-6	93000	13200
OR1-7	93500	13200
OR1-8	75600	10700
OR1-9	91300	12900

Average Cylinder Compressive Strength f'_c = 12100 psi

Population Standard Deviation. σ = 980 psi

Coefficient of Variation V = 8.1%

1 lb. = 4.45 N

1 psi = 6.89 kPa

TABLE 5.6 Load-Strain Data for Specimen 1 (SS1B)

Load No.	Load in lbs.	Strain Readings ($\mu\epsilon$) #1	#2	#3	#4	#5	#6	#7	#8	#9	#10	#11	#12	#13	#14
1	0	0	7	-7	2	4	2	0	0	-3	9	7	2	2	3
2	2970	-22	-22	-28	-13	-4	3	11	18	21	25	15	7	0	-14
3	5925	-44	-47	-7	-23	-3	10	29	21	41	41	25	13	-1	-27
4	8965	-70	-75	5	-34	-5	19	49	-2	59	41	36	15	-6	-42
5	11865	-99	-108	-28	-49	-7	23	64	-15	-16	-9	63	34	-10	-57
6	14835	-129	-139	76	-57	-5	36	59	-18	-20	-66	64	43	-12	-71
7	17665	-164	-185	50	-69	4	46	15	-39	-26	-81	29	60	-8	-86
8	20645	-209	-255	-146	-85	22	70	-31	-49	-25	-102	-40	819	33	-85
9	20000	-210	-253	-149	-80	25	48	-43	-52	-25	-112	-41	93	36	-85
10	23715	-259	-309	-174	-77	37	9	-58	-46	1	-124	-38	1406	45	-100
11	26690	-299	-360	-176	-69	23	-8	-70	-45	47	-161	-43	1895	39	-107
12	29700	-336	-412	-214	-73	8	-18	-78	-49	81	-175	-43	2319	8	-112
13	32710	-372	-468	-289	-70	5	-12	-85	-57	112	-209	-29	2689	-7	-118
14	34800	-393	-501	-314	-74	-5	-23	-91	-64	145	-221	-8	2947	-21	-119
15	36555	-413	-533	-337	-79	-6	-20	-92	-65	170	-224	-14	3131	-26	-123
16	38015	-428	-555	-307	-79	-10	-24	-98	-67	193	-230	-15	3265	-31	-125
17	39790	-448	-584	-323	-83	-12	-25	-99	-69	214	-235	-1	3423	-33	-131
18	42000	-467	-614	-342	-88	-16	-27	-100	-72	240	-246	-2	3581	-37	-136
19	37410	-429	-556	-247	-75	-22	-33	-102	-81	219	-238	4	3343	-43	-123
20	37190	-427	-555	-256	-74	-24	-37	-102	-81	217	-236	3	3317	-44	-122
21	38200	-436	-570	-256	-79	-23	-37	-105	-81	218	-240	3	3345	-45	-128
22	42000	-468	-618	-299	-94	-26	-41	-106	-79	234	-248	1	3521	-44	-142

1 lb. = 4.45 N
1 in. = 25.4 mm.

Table 5.7: Load-Average Strain Data for Specimen 1 (5518)

Load in kips	Deflection in. X 10^{-3}	Avg.Strain at top. in./in.X10^{-6} (gages 1&2)	Avg.Strain at 2 in. in./in.X10^{-6} (gages 3&14)	Avg.Strain at 4 in. in./in.X10^{-6} (gages 4&13)	Avg.Strain at 6 in. in./in.X10^{-6} (gages 5&12)
0.0	0	3	-2	2	3
3.0	68	-22	-21	-6	1
5.9	90	-45	-17	-12	5
9.0	98	-72	-28	-20	5
11.9	109	-104	-43	-30	14
14.8	117	-134	-71	-34	19
17.7	129	-174	-81	-38	32
20.6	143	-232	-116	-40	22
20.0	143	-231	-117	-43	25
23.7	161	-284	-137	-45	37
26.7	177	-329	-142	-54	23
29.7	192	-374	-164	-63	8
32.7	209	-420	-204	-70	5
34.8	218	-447	-216	-74	-5
36.6	226	-473	-230	-79	-6
38.0	234	-492	-216	-79	-10
39.8	243	-516	-227	-83	-12
42.0	661	-540	-239	-88	-16
37.4	661	-493	-185	-75	-22
37.2	661	-491	-189	-74	-24
38.2	661	-503	-192	-79	-23
42.0	668	-543	-221	-94	-26

1 kip = 4.45 N, 1 in. = 25.4 mm

TABLE 5.8 Whittemore Strain Gage Readinga for Specimen 1 (SS1B)

Load in lbs.	Whittemore Strain Gage Readings (10^{-5} in/in)				Average Strain Gage Readinga	
	#1 at 8"	#2 at 10"	#3 at 10"	#4 at 8"	At 8" Depth	At 10" Depth
2970	-	-	-	-	-	-
5925	0	0	0	0	-	-
89655	0	0	0	0	-	-
11865	0	0	0	0	-	-
14835	2	0	10	4	3	5
17665	4	0	13	5	5	7
20645	9	5	14	5	7	10
23715	11	11	19	13	12	15
26690	18	18	31	27	23	25
29700	24	28	38	22	23	33
32710	29	35	46	24	27	41
34800	33	42	56	36	35	49
36555	35	42	58	31	33	50

1 lb. = 4.45 N
1 in. ≈ 25.4 mm.

TABLE 5.9 Load-Strain Data for Specimen 2 (SS2B)

Load No.	Average Load in lbs.	Strain Gage Readings ($\mu\epsilon$)							
		#1	#2	#3	#4	#5	#6	#7	#8
1	0	-11	-13	-12	-18	-14	-16	-14	-8
2	7880	-87	-75	-51	-37	-14	-23	-42	-55
3	16900	-192	-171	-102	-54	-4	-10	-59	-115
4	23820	-318	-280	-137	-39	-40	-49	-28	-167
5	31762	-451	-396	-172	-28	-39	-92	159	-216
6	39640	-574	-495	-200	-20	-64	-106	453	-259
7	47350	-693	-597	-239	-31	-69	-119	578	-297
8	53407	-783	-679	-275	-37	-69	-130	610	-343
9	59159	-869	-765	-316	-45	-73	-140	688	-371
10	0	-	9	5	1	0	1	-3	-3 *
11	59830	-	-794	-368	-38	22	-7	423	-369
12	63850	-	-841	-394	-41	22	-7	446	-390
13	67760	-	-896	-421	-45	20	-11	468	-412
14	71705	-	-947	-451	-50	21	-13	483	-436
15	75750	-	-995	-474	-49	31	-7	493	-461
16	79760	-	-1057	-508	-56	31	-7	508	-488
17	80365	-	-1086	-517	-50	29	-7	557	-494
18	83500	-	-1147	-540	-48	22	-12	585	-525
19	86700	-	-1138	-525	-40	20	-15	507	-552

* The spreader beam failed at 60000 lbs. The beam was reloaded using a new spreader beam. The new no load reading was used in further calculations.

1 lb. = 4.45 N
1 in. = 25.4 mm.

Table 5.10: Load-Average Strain Data for Specimen 2 (552B)

Load in kips	Deflection in. X 10^{-3}	Avg.Strain at top. in./in.X10^{-6} (gages 1&2)	Avg.Strain at 2 in. in./in.X10^{-6} (gages 3&8)	Avg.Strain at 4 in. in./in.X10^{-6} (gages 4&7)	Avg.Strain at 6 in. in./in.X10^{-6} (gages 5&6)
0.0	0	-12	-10	-16	15
7.9	27	-81	-53	-39	-18
16.9	60	-181	-109	-57	-7
23.8	95	-299	-152	-33	-44
31.8	138	-423	-194	-28	-85
31.8	138	-423	-194	-28	-65
39.6	165	-535	-230	-20	-85
47.4	217	-613	-268	-31	-64
53.4	260	-731	-309	-37	-99
59.2	294	-817	-344	-45	-106
0	--	9	1	-1	1
59.8	336	-794	-368	-38	8
63.9	354	-841	-392	-41	8
67.8	374	-896	-417	-45	5
71.7	395	-947	-444	-50	4
75.8	418	-995	-467	-49	12
79.8	430	-1057	-498	-56	12
80.4	460	-1086	-505	-50	11
83.5	472	-1147	-533	-48	5
86.7	495	-1138	-539	-40	3

1 kip = 4.45 N

1 in. = 25.4 mm

TABLE 5.11 Whittemore Strain Gage Readings for Shear Specimen 2 (SS2B)

Load in lbs.	Whittemore Strain Gage Readings (10^{-5} in/in)				Average Strain Gage Readinga	
	#1 at 8"	#2 at 10"	#3 at 10"	#4 at 8"	At 8" Depth	At 10" Depth
0	0	0	0	0	0	0
7880	10	12	1	12	11	7
16900	15	17	6	11	13	12
23820	28	54	18	28	28	36
31762	45	72	30	39	42	51
39640	60	107	48	43	52	78
47350	78	117	67	62	70	87
53407	88	131	81	78	83	106
59159	97	142	96	87	92	119
61000	98	137	100	90	94	119
61750	78	137	105	81	80	121
63125	90	137	112	82	86	125

1 lb. = 4.45 N
1 in. = 25.4 mm.

TABLE 5.12 Load-Strain Data for Specimen 3 (URI)

Load No.	Average Load in lbs.	Strain Gage Readings ($\mu\epsilon$) #1	#2	#3	#4	#5	#6	#7	#8
1	0	4	3	2	-4	-3	-1	-1	-1
2	11810	-98	-137	-60	-28	0	13	-23	-58
3	25000	-280	-375	-116	-5	-18	30	-31	-133
4	35670	-412	-542	-155	5	-22	25	-43	-181
5	47820	-569	-733	-207	1	-36	54	47	-227
6	59520	-710	-911	-258	-6	-36	58	140	-278
7	67570	-809	-1031	-296	-5	-42	57	162	-317
8	75400	-945	-1188	-337	0	-44	66	255	-363
9	83460	-1067	-1337	-386	-2	-45	70	315	-409
10	87450	-1138	1423	-403	1	-45	75	364	-435
11	89200	-1327	-1620	-359	20	-61	91	666	-391
12	85100	-1352	-1653	-357	15	-61	75	561	-385
13	85100	-1545	-1845	-335	3	-74	143	451	-342
14	87760	-1609	-1937	-329	-18	-72	103	387	-321
15	88660	-1786	-2141	-277	-21	-79	182	493	-233
16	91000	-2019	-2402	-209	-30	-82	193	564	-134
17	91510	-2187	-2606	-156	-27	-77	127	531	-121
18	91680	-2198	-2628	-158	-27	-78	109	502	-124
19	90310	-2002	-2420	-287	-25	-72	29	332	-181
20	90800	-1885	-2307	-348	-36	-82	1	161	-272

1 lb. = 4.45 N

Table 5.13: Load-Average Strain Data for Specimen 3 (UR1)

Load in kips	Deflection in. X 10^{-3}	Avg.Strain at top. in./in.X10^{-6} (gages 1&2)	Avg.Strain at 2 in. in./in.X10^{-6} (gages 3&8)	Avg.Strain at 4 in. in./in.X10^{-6} (gages 4&7)	Avg.Strain at 6 in. in./in.X10^{-6} (gages 5&6)
0.0	0	4	1	-3	-2
11.8	48	-118	-59	-25	6
25.0	116	-328	-124	-31	24
35.7	170	-477	-168	-43	1
47.8	230	-651	-217	47	9
59.5	294	-810	-268	140	11
67.6	324	-920	-307	162	8
75.4	378	-1066	-350	255	11
83.5	424	-1202	-398	315	13
87.5	450	-1280	-419	364	15
89.2	478	-1473	-512	666	15
85.1	582	-1502	-371	561	7
85.1	640	-1695	-338	451	35
87.8	685	-1773	-325	387	15
88.7	755	-1964	-255	493	51
91.0	780	-2210	-171	564	55
91.5	780	-2396	-138	531	25
91.7	780	-2413	-141	502	16
90.3	780	-2211	-234	332	-21
90.8	780	-2096	-310	161	-42

1 kip = 4.45 N

1 in. = 25.4 mm

TABLE 5.14 Whittemore Strain Gage Readings for Specimen 3 (UR1)

Load in lbs.	Whittemore Strain Gage Readings (10^{-5} in/in)				Average Strain Gage Readings	
	#1 at 8"	#2 at 10"	#3 at 10"	#4 at 8"	At 8" Depth	At 10" Depth
0	–	0	0	–	–	0
11810	–	14	51	–	–	33
25000	–	57	110	–	–	84
35670	–	84	126	–	–	105
47820	–	110	175	–	–	143
59520	–	124	160	–	–	142
67570	–	121	165	–	–	143
75400	–	147	176	–	–	162
83460	–	158	203	–	–	181
87450	–	160	204	–	–	182

1 lb. = 4.45 N
1 in. = 25.4 mm.

TABLE 5.15 Load-Strain Data for Specimen 4 (OR1)

Load No.	Average Load in lbs.	Strain Gage Readings ($\mu\epsilon$) #1	#2	#3	#4	#5	#6	#7	#8
1	0	-12	10	-11	7	2	-2	-7	9
2	10800	-101	-125	-76	-40	10	2	-39	-77
3	19770	-181	-224	-131	-60	16	12	-61	-140
4	30000	-285	-355	-196	-78	35	36	-89	-220
5	40000	-395	-491	-266	-99	105	148	-117	-303
6	50000	-504	-622	-337	-117	181	368	-140	-381
7	60000	-615	-757	-407	-133	301	524	-165	-464
8	70000	-727	-894	-480	-154	411	649	-194	-547
9	80000	-847	-1041	-560	-176	515	751	-223	-633
10	90000	-967	-1186	-641	-201	618	844	-257	-721
11	99500	-1092	-1335	-726	-226	716	925	-292	-813
12	105620	-1173	-1438	-784	-246	768	963	-314	-873
13	112000	-1250	-1533	-837	-266	813	993	-342	-933
14	118000	-1328	-1634	-896	-285	850	1015	-364	-992
15	124000	-1409	-1733	954	-303	892	1036	-388	-1054
16	136000	-1494	-1843	-1017	-327	925	1048	-414	-1119
17	142000	-1572	-1946	-1075	-345	968	1065	-439	-1179
18	148000	-1658	-2057	-1138	-369	999	1078	-468	-1246
19	154000	-1756	-2189	-1211	-398	1024	1081	-501	-1322
20	160000	-1844	-2303	-1278	-420	1056	1091	-529	-1392
21	166000	-1932	-2417	-1339	-442	1095	1107	-558	-1458
22	170000	-2032	-2551	-1414	-471	1129	1109	-591	-1535
23	172000	-2107	-2658	-1471	-492	1156	1107	-617	-1600
24	174000	-2152	-2729	-1508	-508	1167	1097	-640	-1642
25	176000	-2197	-2799	-1545	-524	1179	1086	-658	-1683
26	178000	-2244	-2880	-1635	-548	1186	1074	-679	-1728
27	180000	-2284	-2925	-1618	-553	1229	1063	-699	-1774
28		-1342	-1721	-2419	-921	401	484	-1012	-2122

1 lb. = 4.45 N
1 in. = 25.4 mm.

Table 5.16: Load-Average Strain Data for Specimen 4 (OR1)

Load in kips	Deflection in. X 10^{-3}	Avg.Strain at top. in./in.X10^{-6} (gages 1&2)	Avg.Strain at 2 in. in./in.X10^{-6} (gages 3&8)	Avg.Strain at 4 in. in./in.X10^{-6} (gages 4&7)	Avg.Strain at 6 in. in./in.X10^{-6} (gages 5&6)
0.0	0	-1	-1	0	0
10.8	132	-113	-77	-39	6
19.8	162	-202	-135	-60	14
30.0	200	-320	-208	-84	35
40.0	232	-443	-285	-108	127
50.0	268	-563	-359	-129	274
60.0	306	-659	-435	-149	412
70.0	342	-810	-513	-174	530
80.0	378	-944	-596	-199	383
90.0	418	-1078	-681	-229	731
99.5	440	-1214	-769	-259	821
105.6	461	-1305	-828	-280	866
112.0	486	-1392	-885	-304	903
118.0	504	-1481	-994	-325	923
124.0	532	-1571	-1004	-346	964
136.0	558	-1668	-1068	-371	987
142.0	583	-1759	-1127	-392	1017
148.0	583	-1858	-1192	-419	1038
154.0	583	-1973	-1335	-475	1073
160.0	583	-2073	-1335	-475	1073
166.0	583	-2174	-1399	-500	1101
170.0	583	-2292	-1474	-531	1119

(Table 5.16 continued)

Load in kips	Deflection in. X 10^{-3}	Avg.Strain at top. in./in.X10^{-6} (gages 1&2)	Avg.Strain at 2 in. in./in.X10^{-6} (gages 3&8)	Avg.Strain at 4 in. in./in.X10^{-6} (gages 4&7)	Avg.Strain at 6 in. in./in.X10^{-6} (gages 5&6)
172.0	583	-2383	-1535	-555	1132
174.0	583	-2441	-1575	-574	1132
176.0	583	-2498	-1614	-591	1132
178.0	583	-2562	-1681	-610	1130
180.0	583	-2604	-1696	-626	1146

1 kip = 4.45 N

1 in. = 25.4 mm

Table 5.17: Stress strain relation for 3 in. X 6 in. cylinder for Specimen-1(SS18)

Load in lbs.	Stress psi	Longitudinal Strain Readings U6 Gage #1	Gage #2	Average Longitudinal Strain Readings U6
0	0	0	-1	1
5000	700	-37	-165	-101
10000	1400	-164	-243	-203
15000	2100	-297	-328	-313
20000	2800	-413	-421	-417
25000	3500	-535	-524	-529
30000	4200	-650	-623	-637
33000	4700	-728	-689	-708
36000	5100	-804	-755	-779
39000	5500	-882	-823	-852
42000	5900	-952	-886	-919
45000	6400	-1024	-951	-988
48000	6800	-1103	-1024	-1064
52000	7400	-1202	-1116	-1159
55100	7800	-1283	-1192	-1238
58000	8200	-1369	-1275	-1322
61000	8600	-1442	-1348	-1395
64000	9100	-1518	-1425	-1472
67400	9500	-1611	-1522	-1567
70000	9900	-1685	-1597	-1641

(Table 5.17 continued)

Load in lbs.	Stress psi	Longitudinal Strain Readings UG Gage #1	Gage #2	Average Longitudinal Strain Readings UG
73000	10300	-1787	-1710	-1747
76000	10800	-1884	-1811	-1847
78200	11100	-1953	-1889	-1921
81000	11500	-2058	-2014	-2037

1 lb = 4.45 N

1 psi= 6.89 kPa

TABLE 5.18 Stress-Strain Relation and Poisson's Ratio for 3 in. x 6 in. Cylinder for Specimen 2 (SS2B)

Load in lbs.	Stress psi.	Longitudinal (με) Strain Gage Readings #1	#2	Transverse (με) Strain Gage Readings #3	#4	Average Longitudinal Strain (με)	Average Transverse Strain (με)	Poisson's Ratio
0	0	2	1	3	3	1	3	-
2000	280	-50	-50	28	3	-50	16	0.320
5000	710	-89	-128	37	5	-109	21	0.192
10000	1400	-192	-261	50	17	-227	34	0.149
15000	2100	-294	-383	58	31	-339	44	0.131
20000	2820	-392	-507	65	45	-450	55	0.122
25000	3540	-491	-640	78	59	-506	69	0.121
30000	4240	-586	-760	92	71	-673	82	0.121
33000	4670	-650	-835	100	79	-743	90	0.121
36000	5090	-712	-915	109	88	-814	99	0.121
39000	5520	-779	-995	119	98	-887	109	0.122
42000	5940	-838	-1065	125	106	-952	116	0.121
45000	6370	-907	-1148	136	115	-1028	126	0.122
48000	6790	-968	-1222	143	124	-1095	134	0.122
51000	7220	-1035	-1303	151	137	-1169	144	0.123
54000	7640	-1102	-1385	163	145	-1244	154	0.123
57000	8060	-1169	-1466	175	159	-1318	167	0.126
60000	8490	-1242	-1550	184	171	-1396	178	0.128
63000	8910	-1319	-1638	192	183	-1479	188	0.127
66000	9340	-1395	-1727	203	196	-1561	200	0.128
69000	9760	-1482	-1824	214	221	-1653	218	0.132
72000	10190	-1581	-1932	225	230	-1756	228	0.130
75000	10610	-1678	-2011	236	213	-1855	225	0.121
77000	10890	-1759	-2117	247	171	-1938	209	0.108
79000	11180	-1845	-2205	257	150	-2025	204	0.101
81000	11460	-1931	-2297	269	135	-2114	202	0.096
83000	11740	-2018	-2394	286	140	-2206	213	0.097
85000	12030	-2114	-2499	321	217	-2307	279	0.121
87000	12310	-2240	-2714	326	-	-2477	-	-
87200		--Failure--						

1 lb. = 4.45 N

Table 5.19: Load and Stress Data for Shear Specimen I (S5IB)
Using Cylinder Stress-Strain Curve 2

Load in kips	Neutral axis Depth in.	Stress at top,psi	Stress at 2 in.,psi	Stress at 4 in.,psi	Stress at 6 in.,psi
3.0	6.46	-170	-130	-70	-30
5.9	5.63	-310	-110	-100	30
9.0	5.64	-480	-180	-150	30
11.9	5.68	-670	-260	-210	80
14.8	5.44	-860	-440	-240	110
17.7	5.17	-1110	-500	-260	190
20.6	4.88	-1470	-710	-270	140
20.0	4.94	-1460	-720	-290	150
23.7	4.77	-1790	-850	-300	220
26.7	4.80	-2080	-880	-360	140
29.7	4.83	-2360	-1010	-410	50
32.7	4.82	-2650	-1260	-460	30
34.8	6.23	-2820	-1340	-480	-70
36.6	6.24	-2990	-1420	-510	-70
38.0	6.37	-3100	-1340	-510	-100
39.8	6.41	-3260	-1410	-540	-110
42.0	6.55	-3410	-1480	-570	-140
37.4	6.97	-3110	-1140	-490	-170
37.2	7.12	-3100	-1170	-480	-180
38.2	6.95	-3180	-1190	-510	-180
42.0	6.87	-3430	-1370	-600	-200

1 kip = 4.45 kN, 1 psi = 6.89 kPa, 1 in = 25.4 mm

Table. 5.20: Actual and Calculated Moments using Triangular and Parabolic Stress Blocks for SS1B

Load no.	Load in kips	Mu (test) kip-ft	Mu (^ calc) kip-ft	Mu(~calc) kip-ft	Mu (test) / Mu (^ calc)	Mu(test) / Mu(~calc)
2	3.0	3.5	3.1	3.3	1.11	1.04
3	5.9	6.9	5.1	4.1	1.35	1.67
4	9.0	10.5	7.9	6.5	1.32	1.61
5	11.9	13.8	11.2	9.2	1.24	1.51
6	14.8	17.3	13.8	12.7	1.25	1.36
7	17.7	20.6	17.2	15.0	1.20	1.38
8	20.6	24.1	21.6	19.7	1.11	1.22
9	20.0	23.3	21.7	19.9	1.07	1.17
10	23.7	27.7	25.9	23.5	1.07	1.18
11	26.7	31.1	30.3	26.0	1.03	1.20
12	29.7	34.7	34.5	29.7	1.00	1.17
13	32.7	38.2	38.6	34.9	0.99	1.09
14	34.8	40.6	50.4	38.3	0.81	1.06
15	36.6	42.6	53.5	40.6	0.80	1.05
16	38.0	44.4	56.4	40.5	0.79	1.10
17	39.8	46.4	59.6	42.6	0.78	1.09
18	42.0	49.0	63.3	44.8	0.77	1.09
19	37.4	43.6	60.4	38.1	0.72	1.14
20	37.2	43.4	61.2	38.3	0.71	1.13
21	38.2	44.6	61.7	39.4	0.72	1.13
22	42.0	49.0	66.0	44.1	0.74	1.11

1 kip = 4.45 N

1 kip-ft= 1.36 kN-m

Table 5.21: Actual and Calculated Deflections for SS1B

Load in kips	Actual Def. in.	Cal.Def. in.	Actual Def. / Cal.Def.
3.0	0.068	0.004	15.114
5.9	0.090	0.009	10.019
9.0	0.098	0.014	7.212
11.9	0.109	0.018	6.062
14.8	0.117	0.050	2.345
17.7	0.129	0.059	2.171
20.7	0.143	0.069	2.060
20.0	0.143	0.067	2.127
23.7	0.161	0.080	2.019
26.7	0.177	0.090	1.973
29.7	0.192	0.100	1.923
32.7	0.209	0.110	1.901
34.8	0.218	0.117	1.863
36.6	0.226	0.123	1.839
38.0	0.234	0.128	1.831
39.8	0.243	0.134	1.817
42.0	0.661	0.141	4.681

1 kip = 4.45 N

1 in. = 25.4 mm

Table 5.22: Load and Stress Data for Shear Specimen II (S52B)
Using Cylinder Stress-Strain Curve 2

Load in kips	Neutral axis Depth in.	Stress at top,psi	Stress at 2 in.,psi	Stress at 4 in.,psi	Stress at 6 in.,psi
7.9	6.01	-440	-280	-160	-220
6.9	5.26	-1060	-620	-270	-150
3.8	4.26	-1790	-890	-120	-380
1.8	4.12	-2580	-1150	-90	-630
1.8	4.12	-2580	-1150	-90	-510
9.6	4.03	-3280	-1370	-40	-630
7.4	4.10	-3780	-1610	-110	-500
3.4	4.12	-4520	-1870	-140	-720
9.2	4.15	-5050	-2090	-200	-760
9.8	4.11	-4900	-2240	-150	-60
3.9	4.20	-5330	-2460	-260	70
7.8	4.21	-5660	-2620	-290	50
1.7	4.22	-5970	-2790	-320	40
5.8	4.20	-6260	-2940	-310	90
9.8	4.22	-6620	-3130	-350	90
0.4	4.19	-6790	-3180	-320	90
3.5	4.17	-7150	-3350	-310	50
6.7	4.14	-7100	-3390	-250	30

1 kip = 4.45 kN

1 psi = 6.89 kPa

1 in = 25.4 mm

Table. 5.23: Actual and Calculated Moments Using Triangular and Parabolic Stress Blocks for SS2B

Load no.	Load in kips	Mu (test) kip-ft	Mu (^ calc) kip-ft	Mu(~calc) kip-ft	Mu (test) / Mu (^ calc)	Mu(test) / Mu(~calc)
2	7.9	9.2	7.7	7.6	1.20	1.21
3	16.9	19.7	16.6	16.3	1.19	1.21
4	23.8	27.8	23.5	22.7	1.18	1.22
5	31.8	37.1	33.0	30.5	1.12	1.22
6	31.8	37.1	33.0	30.5	1.12	1.22
7	39.6	46.2	41.2	37.2	1.12	1.24
8	47.4	55.2	48.2	43.5	1.15	1.27
9	53.4	62.3	57.8	51.3	1.08	1.21
10	59.2	69.0	65.0	57.5	1.06	1.20
11	59.8	69.8	62.6	58.7	1.11	1.19
12	63.9	74.5	69.2	64.6	1.08	1.15
13	67.8	79.1	73.7	68.7	1.07	1.15
14	71.7	83.7	77.9	73.0	1.07	1.15
15	75.8	88.4	81.4	76.6	1.09	1.15
16	79.8	93.1	86.4	81.4	1.08	1.14
17	80.4	93.8	88.1	82.9	1.06	1.13
18	83.5	97.4	92.4	87.1	1.05	1.12
19	86.7	101.2	91.2	87.2	1.11	1.16

1 kip = 4.45 kN

1 kip-ft = 1.36 kN-m

Table 5.24: Actual and Calculated Deflections for 5S2B

Load in kips	Actual Def. in.	Cal.Def. in.	Actual Def. / Cal.Def.
7.9	0.027	0.011	2.471
16.9	0.060	0.052	1.161
23.8	0.095	0.073	1.306
31.8	0.138	0.097	1.419
31.8	0.138	0.097	1.419
39.6	0.165	0.121	1.363
47.4	0.217	0.145	1.497
53.4	0.260	0.163	1.593
59.2	0.294	0.181	1.624
59.8	0.336	0.183	1.838
63.9	0.354	0.195	1.812
67.8	0.374	0.207	1.804
71.7	0.395	0.219	1.802
75.8	0.418	0.232	1.804
79.8	0.430	0.244	1.762
80.4	0.460	0.246	1.871
83.5	0.472	0.255	1.849
86.7	0.495	0.265	1.867

1 kip = 4.45 N

1 in. = 25.4 mm

Table 5.25: Load and Stress Data for Under-Reinforced Specimen I (UR1) Using Cylinder Stress-Strain Curve 2

Load in kips	Neutral Axis Depth in.	Stress at top, psi	Stress at 2 in., psi	Stress at 4 in., psi	Stress at 6 in., psi
11.8	4.99	-760	-390	-140	40
25.0	4.57	-2070	-790	-190	180
35.7	4.61	-3020	-1060	-260	40
47.8	3.62	-4110	-1360	330	90
59.5	3.30	-5110	-1680	900	100
67.6	3.30	-5780	-1930	1030	80
75.4	3.15	-6650	-2200	1610	100
83.5	3.11	-7430	-2500	1990	110
87.5	3.07	-7870	-2630	2300	120
89.2	2.87	-8890	-3220	4200	120
85.1	2.79	-9030	-2330	3540	70
85.1	2.86	-9940	-2120	2850	240
87.8	2.91	-10300	-2040	2440	120
88.7	2.68	-11000	-1600	3110	340
91.0	2.47	-11800	-1080	3560	370
91.5	2.41	-11500	-870	3350	180
91.7	2.44	-12200	-890	3170	120
90.3	2.83	-11800	-1470	2100	160
90.8	3.31	-11500	-1950	1020	280
87.5	3.46	-7870	-2630	960	120

1 kip = 4.45 kN

1 psi = 6.89 kPa

Table 5.26 : Actual and Calculated Moments Using Triangular and Parabolic Stress Blocks for UR1

Load no.	Load in kips	Mu (test) kip-ft	Mu (^ calc) kip-ft	Mu(~calc) kip-ft	Mu (test) / Mu (^ calc)	Mu(test) / Mu(~calc)
2	11.8	13.8	11.4	10.4	1.21	1.32
3	25.0	29.2	28.9	23.0	1.01	1.27
4	35.7	41.6	42.5	32.1	0.98	1.29
5	47.8	55.8	47.1	40.1	1.19	1.39
6	59.5	69.4	54.0	49.3	1.29	1.41
7	67.6	78.8	61.0	56.2	1.29	1.40
8	75.4	88.0	67.4	64.2	1.31	1.37
9	83.5	97.4	74.4	72.3	1.31	1.35
10	87.5	102.0	77.8	76.3	1.31	1.34
11	89.2	104.1	82.7	90.1	1.26	1.16
12	85.1	99.3	82.1	78.7	1.21	1.26
13	85.1	99.3	92.1	80.1	1.08	1.24
14	87.8	102.4	97.1	80.7	1.05	1.27
15	88.7	103.4	96.3	78.8	1.07	1.31
16	91.0	106.2	95.7	76.3	1.11	1.39
17	91.5	106.8	91.5	71.7	1.17	1.49
18	91.7	107.0	97.9	75.3	1.09	1.42
19	90.3	105.4	108.3	80.5	0.97	1.31
20	90.8	105.9	121.6	85.5	0.87	1.24
21	87.5	102.0	86.6	76.8	1.18	1.33

1 kip = 4.45 kN

1 kip-ft = 1.36 kN-m

Table 5.27 : Actual and Calculated Moments Using Rectangular and Parabolic Stress Blocks for UR1

Load no.	Load in kips	Mu (test) kip-ft	Mu ([]calc) kip-ft	Mu(~calc) kip-ft	Mu (test) / Mu ([]calc)	Mu(test) / Mu(~calc)
18	91.7	107.0	104.0	75.3	1.03	1.42

1 kip = 4.45 kN

1 kip-ft = 1.36 kN-m

Table 5.28 : Actual and Calculated Moments Using Rectangular and Triangular Stress Blocks for UR1

Load no.	Load in kips	Mu (test) kip-ft	Mu ([]calc) kip-ft	Mu(^calc) kip-ft	Mu (test) / Mu ([]calc)	Mu(test) / Mu(^calc)
18	91.7	107.0	103.97	97.9	1.03	1.09

1 kip = 4.45 kN

1 kip-ft = 1.36 kN-m

Table 5.29: Actual and Calculated Deflections for UR1

Load in kips	Actual Def. in.	Cal.Def. in.	Actual Def. / Cal.Def.
11.8	0.048	0.036	1.335
25.0	0.116	0.076	1.523
35.7	0.170	0.109	1.563
47.8	0.230	0.146	1.579
59.5	0.294	0.181	1.622
67.6	0.324	0.206	1.573
75.4	0.378	0.230	1.645
83.5	0.424	0.254	1.667
87.5	0.450	0.267	1.688
89.2	0.478	0.272	1.759
85.1	0.582	0.259	2.245
85.1	0.640	0.259	2.468
87.8	0.685	0.268	2.561
88.7	0.755	0.270	2.794
91.0	0.780	0.277	2.813

1 kip = 4.45 N

1 in. = 25.4 mm

Table 5.30: Load and Stress Data for Over-Reinforced Specimen (OR1) Using Cylinder Stress-Strain Curve 2

Load in kips	Neutral Axis Depth in.	Stress at top,psi	Stress at 2 in.,psi	Stress at 4 in.,psi	Stress at 6 in.,psi
10.8	5.74	-700	-480	-250	50
19.8	5.62	-1260	-840	-380	100
30.0	5.40	-2000	-1290	-530	230
40.0	4.92	-2770	-1770	-680	800
50.0	4.64	-3530	-2240	-800	1720
60.0	4.53	-4130	-2720	-930	2580
70.0	4.49	-5080	-3220	-1080	3330
80.0	4.68	-5890	-3740	-1240	2400
90.0	4.48	-6690	-4270	-1430	4590
99.5	4.48	-7470	-4820	-1620	5150
105.6	4.49	-7980	-5190	-1750	5420
112.0	4.50	-8440	-5530	-1900	5650
118.0	4.52	-8900	-6190	-2030	5770
124.0	4.53	-9350	-6250	-2160	6020
136.0	4.55	-9800	-6630	-2320	6150
142.0	4.56	-10200	-6970	-2460	6330
148.0	4.57	-10600	-7350	-2620	6460
154.0	4.61	-11000	-8140	-2980	6670
160.0	4.61	-11400	-8140	-2980	6670
166.0	4.62	-11700	-8480	-3140	6830
170.0	4.64	-12000	-8870	-3330	6940
172.0	4.66	-12100	-9170	-3480	7010
174.0	4.67	-12200	-9360	-3600	7010

(Table 5.30 continued)

Load in kips	Neutral Axis Depth in.	Stress at top,psi	Stress at 2 in.,psi	Stress at 4 in.,psi	Stress at 6 in.,psi
176.0	4.69	-12300	-9550	-3710	7010
178.0	4.70	-12400	-9860	-3830	7000
180.0	4.71	-12400	-9920	-3930	7090

1 kip = 4.45 kN

1 psi = 6.89 kPa

Table 5.31 : Actual and Calculated Moments Using Triangular and Parabolic Stress Blocks for OR1

Load no.	Load in kips	Mu (test) kip-ft	Mu (^ calc) kip-ft	Mu(~calc) kip-ft	Mu (test) / Mu (^ calc)	Mu(test) / Mu(~calc)
2	10.8	13	10	10	1.27	1.20
3	19.8	23	18	18	1.31	1.28
4	30.0	35	27	27	1.29	1.27
5	40.0	47	35	37	1.34	1.25
6	50.0	58	42	47	1.38	1.25
7	60.0	70	49	56	1.44	1.25
8	70.0	82	59	67	1.37	1.22
9	80.0	93	71	78	1.31	1.20
10	90.0	105	78	88	1.34	1.19
11	99.5	116	87	99	1.33	1.17
12	105.6	123	93	107	1.32	1.15
13	112.0	131	99	114	1.32	1.15
14	118.0	138	105	125	1.32	1.11
15	124.0	145	110	128	1.31	1.13
16	136.0	159	116	135	1.37	1.17
17	142.0	166	121	142	1.37	1.17
18	148.0	173	126	149	1.37	1.16
19	154.0	180	132	163	1.37	1.11
20	160.0	187	136	164	1.37	1.14
21	166.0	194	140	171	1.38	1.14
22	170.0	198	144	178	1.38	1.12
23	172.0	201	146	183	1.38	1.10
24	174.0	203	147	186	1.38	1.09

(Table 5.31 continued)

Load no.	Load in kips	Mu (test) kip-ft	Mu (^ calc) kip-ft	Mu(~calc) kip-ft	Mu (test) / Mu (^ calc)	Mu(test) / Mu(~calc)
25	176.0	205	149	189	1.38	1.08
26	178.0	208	150	194	1.38	1.07
27	180.0	210	151	195	1.39	1.07

1 kip = 4.45 kN.

1 kip-ft = 1.36 kN-m

Table 5.32 : Actual and Calculated Moments Using Rectangular and Parabolic Stress Blocks for OR1

Load no.	Load in kips	Mu (test) kip-ft	Mu ([]calc) kip-ft	Mu(~calc) kip-ft	Mu (test) / Mu ([]calc)	Mu(test) / Mu(~calc)
27	180.0	210	163	195	1.29	1.07

1 kip = 4.45 kN.

1 kip-ft = 1.36 kN-m

Table 5.33 : Actual and Calculated Moments Using Rectangular and Triangular Stress Blocks for OR1

Load no.	Load in kips	Mu (test) kip-ft	Mu ([]calc) kip-ft	Mu(^calc) kip-ft	Mu (test) / Mu ([]calc)	Mu(test) / Mu(^calc)
27	180.0	210	163	151	1.29	1.39

1 kip = 4.45 kN

1 kip-ft = 1.36 kN-m

Table 5.34: Actual and Calculated Deflections for OR1

Load in kips	Actual Def. in.	Cal.Def. in.	Actual Def. / Cal.Def.
10.8	0.132	0.014	9.106
19.8	0.162	0.038	4.272
30.0	0.200	0.057	3.481
40.0	0.232	0.077	3.029
50.0	0.268	0.096	2.799
60.0	0.306	0.115	2.663
70.0	0.342	0.134	2.551
80.0	0.378	0.153	2.467
90.0	0.418	0.172	2.425
99.5	0.440	0.191	2.309
105.6	0.461	0.202	2.280
112.0	0.486	0.214	2.266
118.0	0.504	0.226	2.230
124.0	0.532	0.237	2.240
136.0	0.558	0.260	2.142
142.0	0.583	0.272	2.144

1 kip = 4.45 N

1 in. = 25.4 mm

Table 5.35: Shear Stress Values (Actual and Calculated) for Specimens SS1B and SS2B

Specimen	f'_c	Measured Shear Force lbs		Predicted Shear Force lbs			
				ACI equations		Zsutty's	
		Cracking	Ultimate	11-3	11-6	Crack.	Ultimate
SS1B	9500	19000	21000	16700	17600	21400	23000
SS2B	11400	--	19650	18300	19000	22700	24400

1 lb. = 4.45 N

1 psi = 6.89 kPa

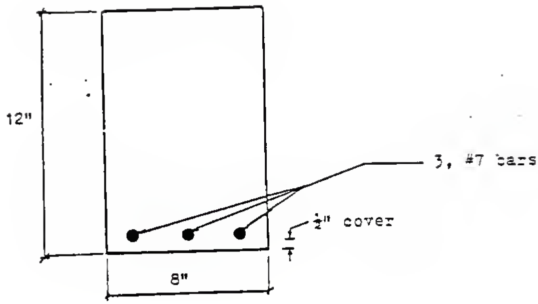


Fig. 3.1: An Arbitrary Section with Reinforcing Bar Arrangement Near Mid-span for Shear Specimen I (SS1B) (1 in. = 25.4 mm)

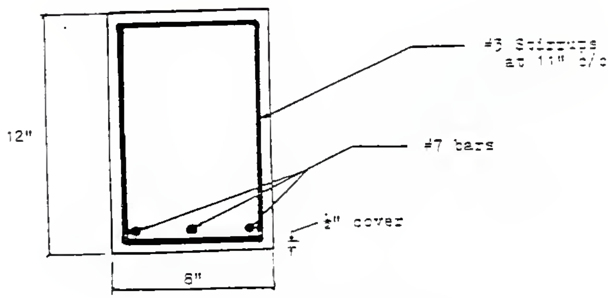


Fig. 3.2: An Arbitrary Section with Reinforcing Bar Arrangement Near Mid-span for Shear Specimen II (SS2B) (1 in. = 25.4 mm)

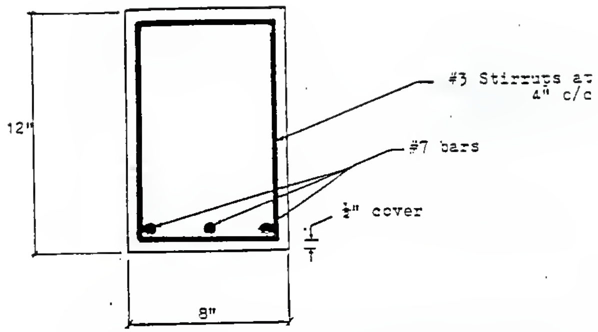


Fig. 3.3: An Arbitrary Section with Reinforcing Bar Arrangement Near Mid-span for Under-reinforced Specimen (UR1) (1 in. = 25.4 mm)

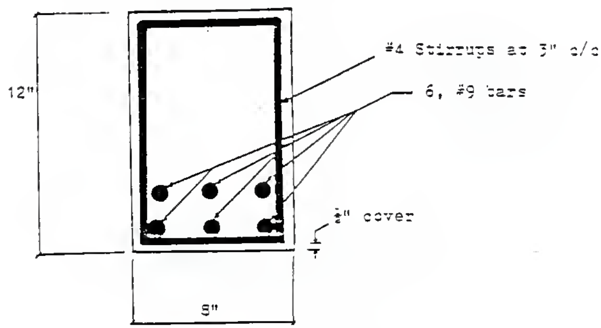


Fig. 3.4: An Arbitrary Section with Reinforcing Bar Arrangement Near Mid-span for Over-reinforced Specimen (OR1) (1 in. = 25.4 mm)

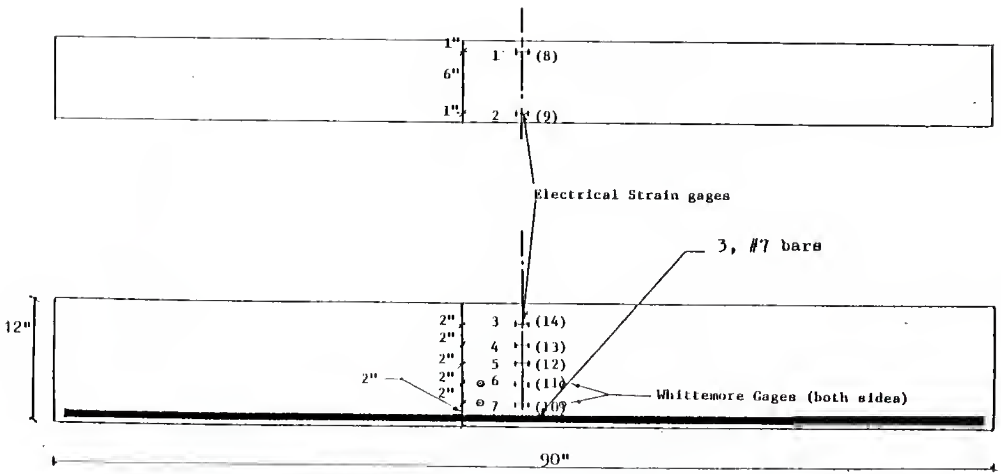


Fig. 3.5: Reinforcement Layout for Specimen 1 (SS1B)
(1 in. = 25.4 mm)

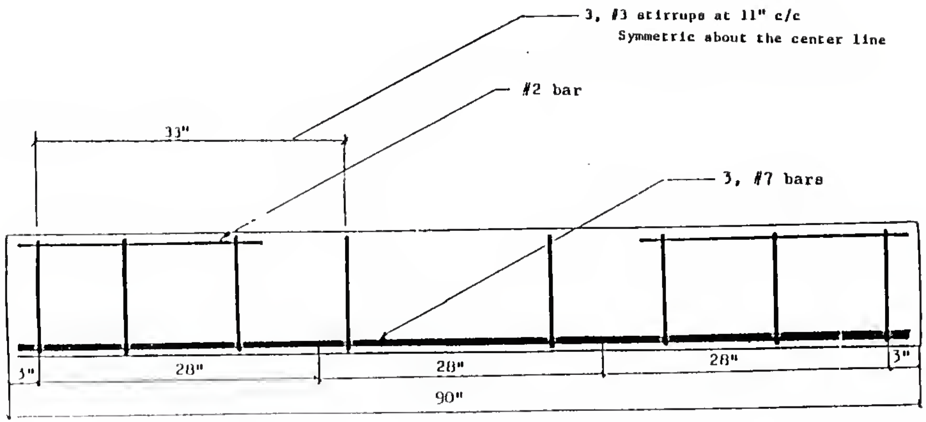


Fig. 3.6: Reinforcement Layout for Shear Specimen 2 (SS2B)
(1 in. = 25.4 mm)

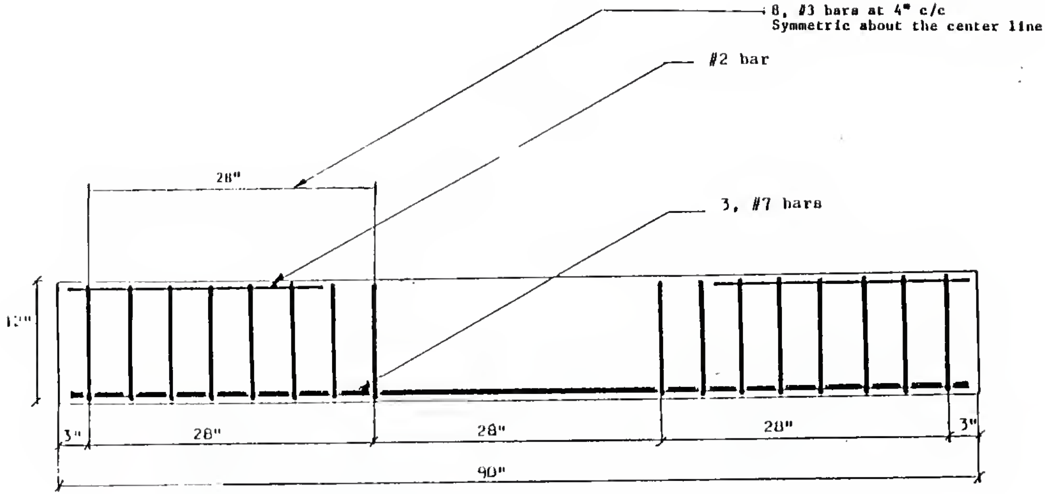


Fig. 3.7: Reinforcement Layout for Under-reinforced Specimen (UR1)
(1 in. = 25.4 mm)

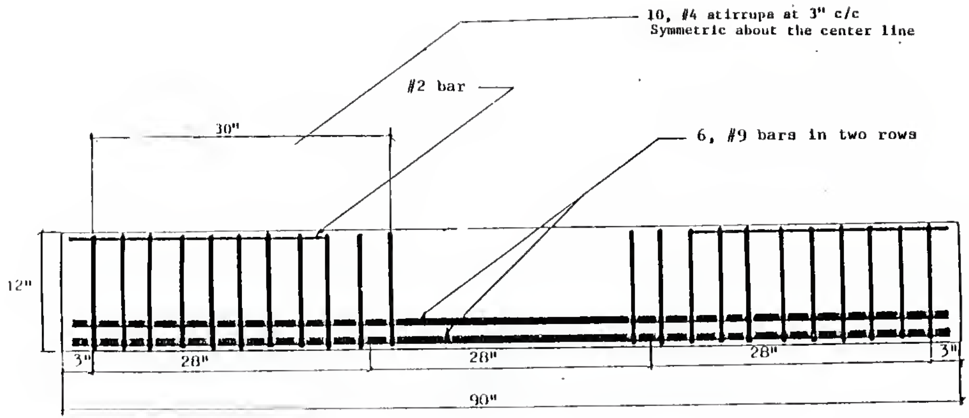


Fig. 3.8: Reinforcement Layout for Over-reinforced Specimen (OR1)
(1 in. = 25.4 mm)

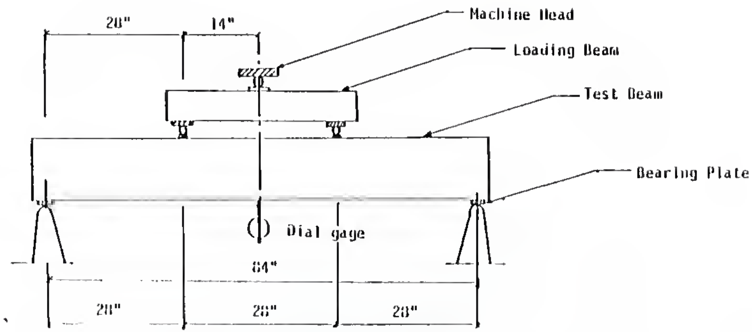


Fig. 3.9: Test Setup and Loading Arrangement
(1 in. = 25.4 mm)

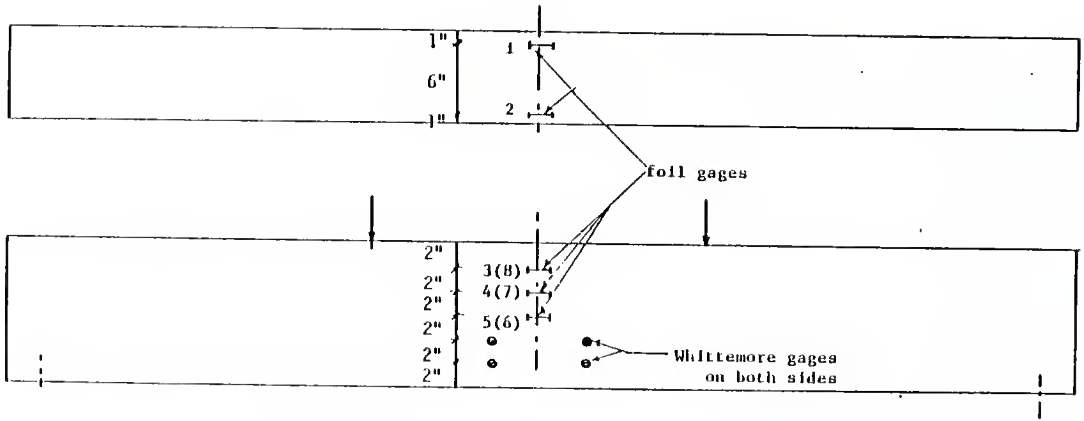


Fig. 3.10: Strain Gage Locations

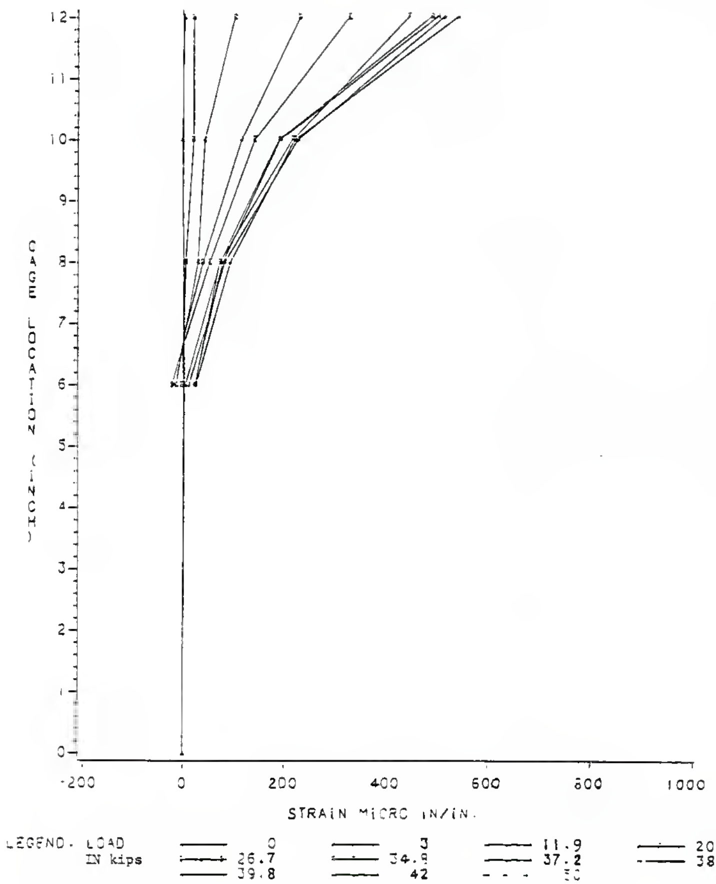


Fig. 5.1: Location of Strain Gages vs. Strain for Shear Specimen I (SS1B)
(corresponding to Table 5.2)
(1 kip = 4.45 kN)

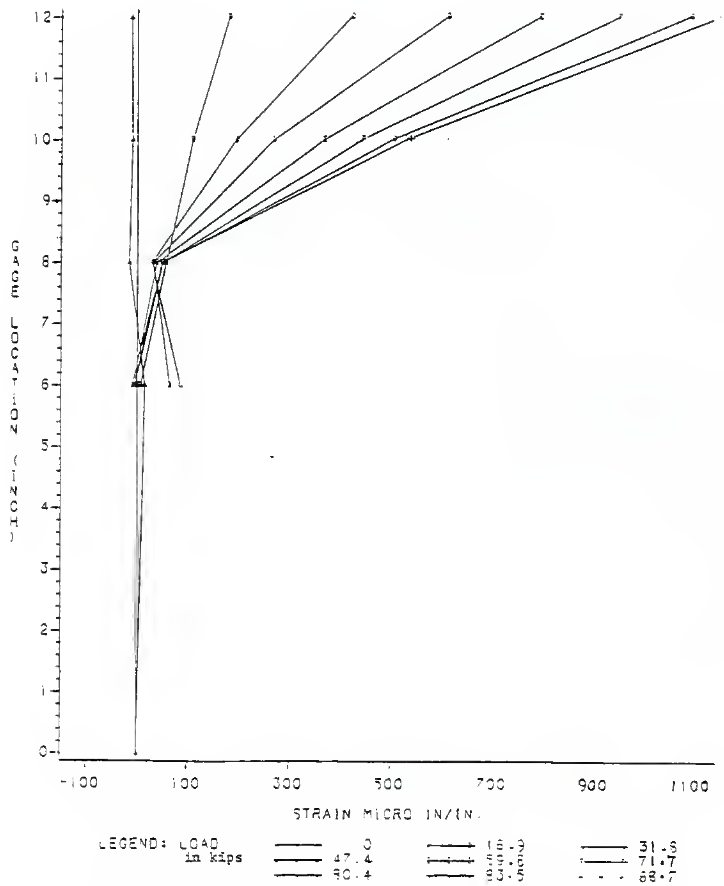


Fig. 5.2: Location of Strain Gages vs. Strain for Shear Specimen II (SS22)
(corresponding to Table 5.4)
(1 kip = 4.45 kN)

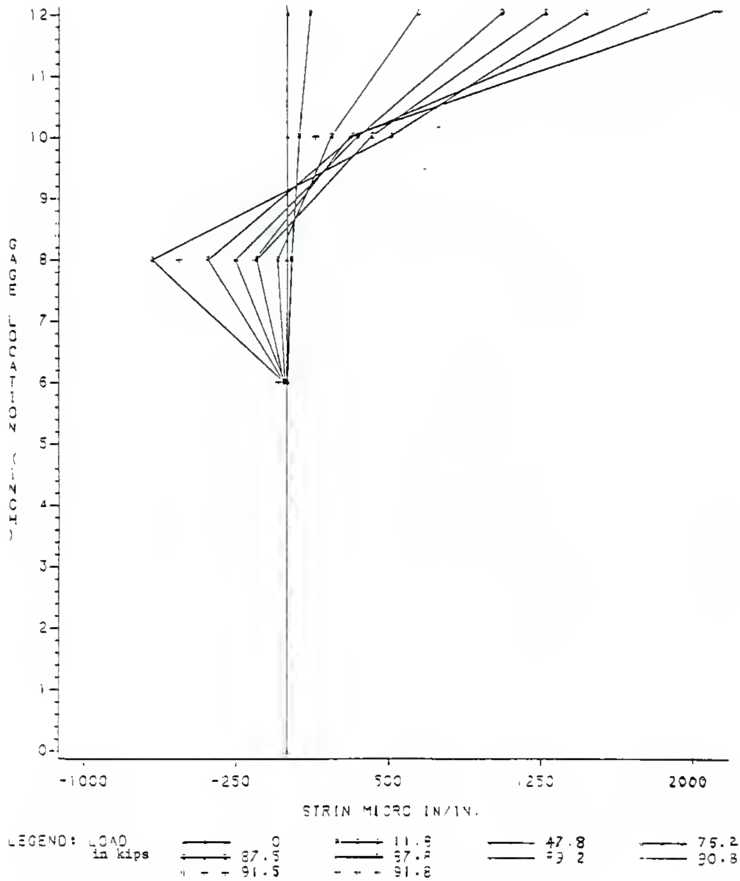


Fig. 5.3: Location of Strain Gages vs. Strain for Under-reinforced Specimen (UR1)
(corresponding to Table 5.6)
(1 kip = 4.45 kN)

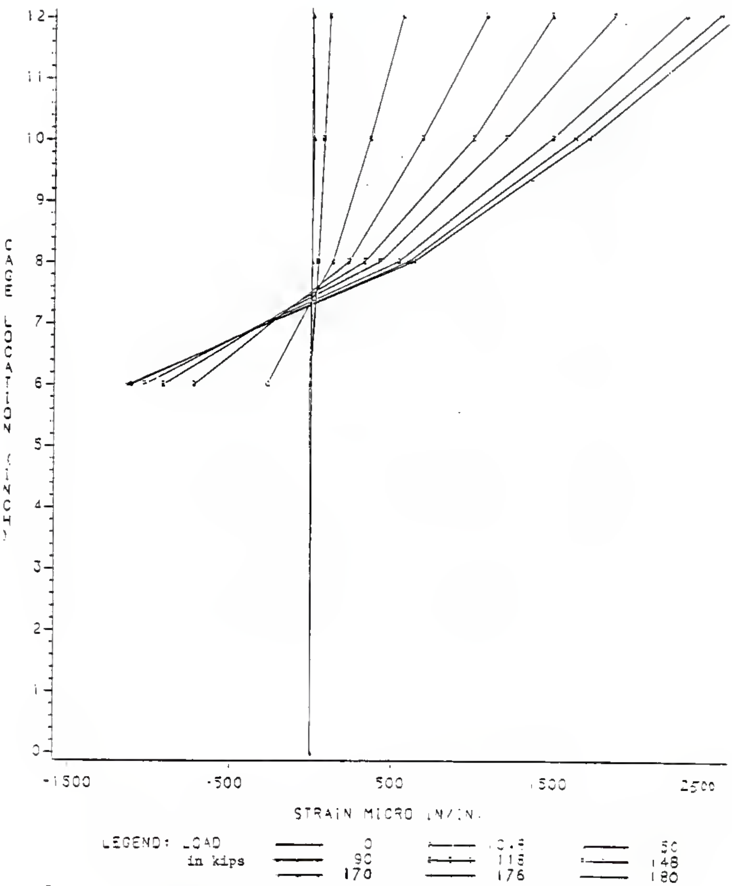


Fig. 5.4: Location of Strain Gages vs. Strain for Over-reinforced Specimen (OR1)
(corresponding to Table 5.8)
(1 kip = 4.45 kN)

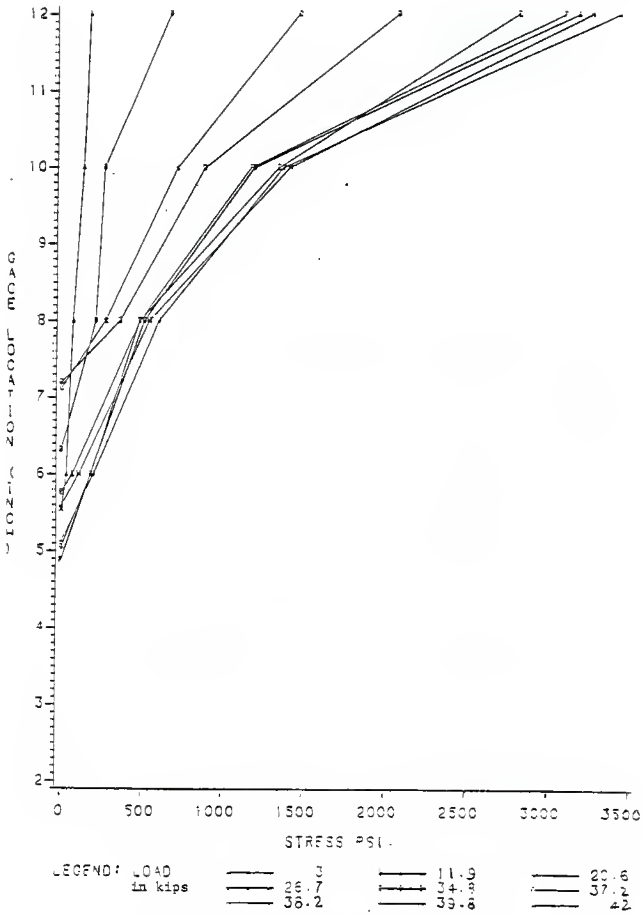


Fig. 5.5: Stress Block of Shear Specimen I (SS1B)
(corresponding to Table 5.19)
(1 in. = 25.4 mm, 1 psi = 6.89 kPa)

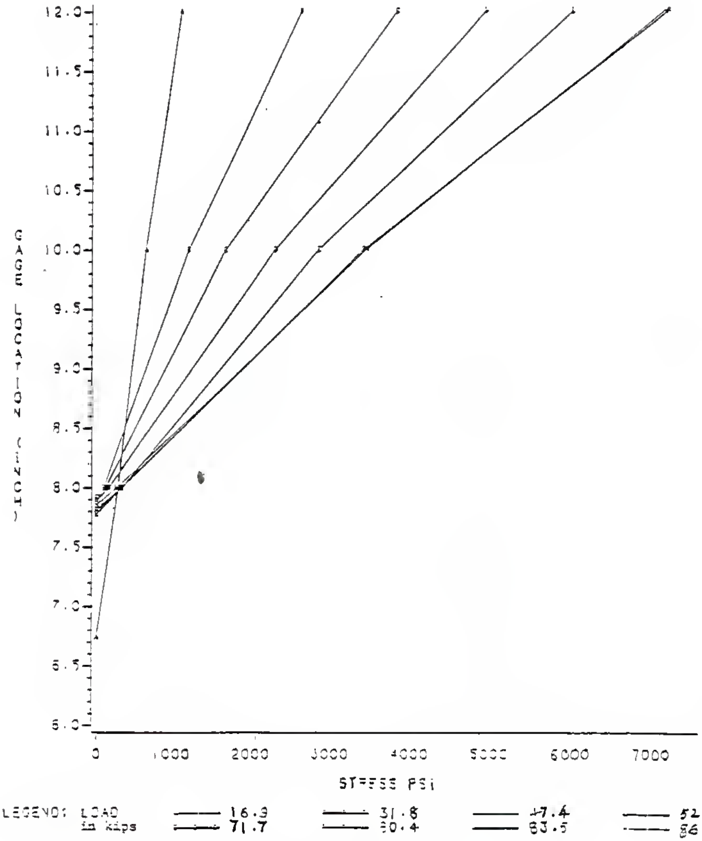


Fig. 5.5: Stress Block of Shear Specimen II (SS2E)
(corresponding to Table 5.22)
(1 in. = 25.4 mm, 1 psi = 6.89 kPa)

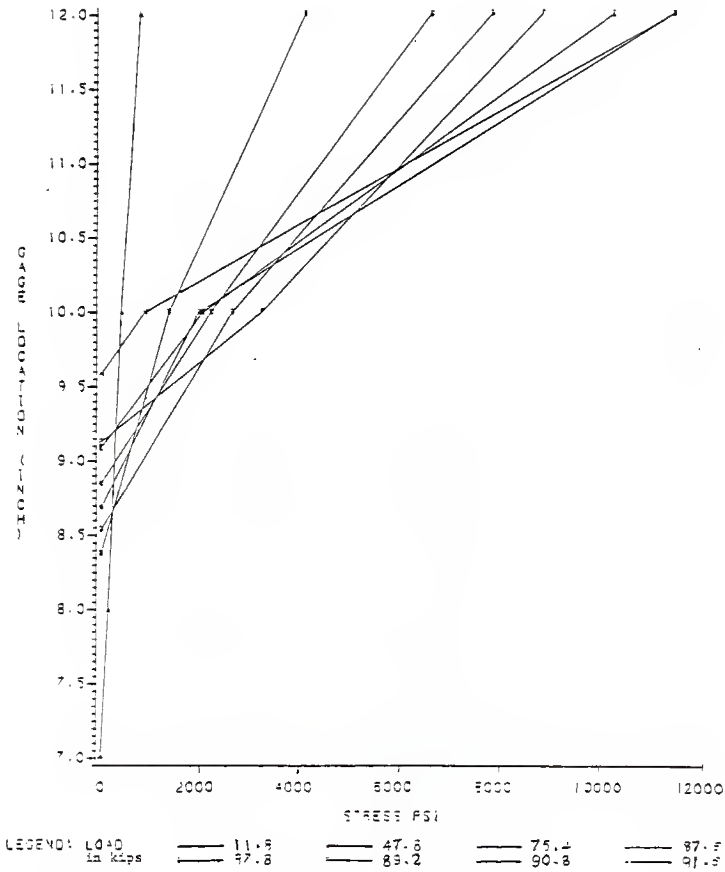


Fig. 5.7: Stress Block of Under-reinforced Specimen (UR1)
(corresponding to Table 5.25)
(1 in. = 25.4 mm, 1 psi = 6.89 kPa)

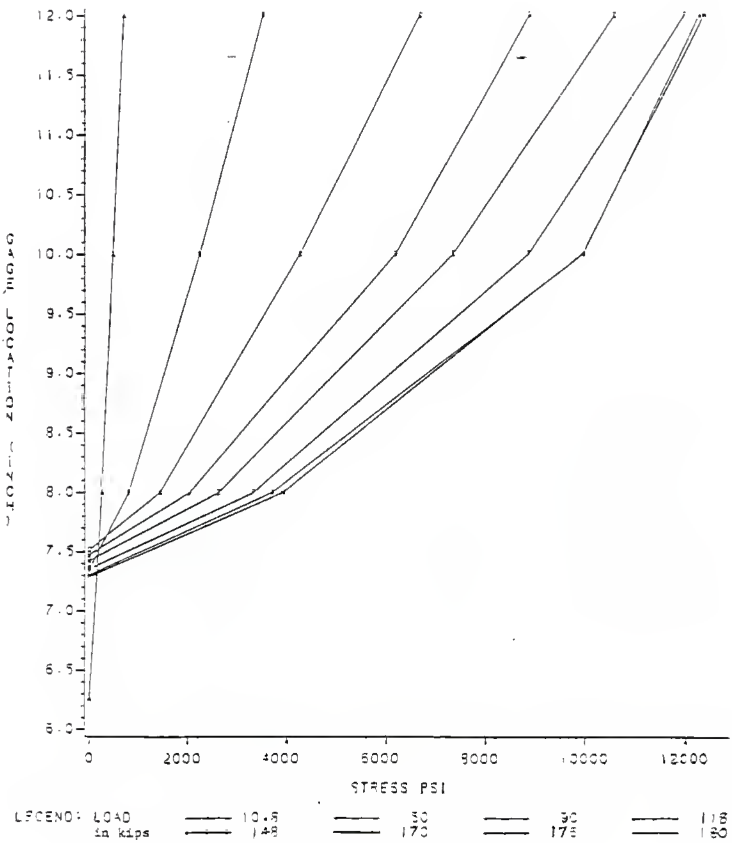


Fig. 5.8: Stress Block of Over-reinforced Specimen (OR1)
(corresponding to Table 5.30)
(1 in. = 25.4 mm, 1 psi = 6.89 kPa)

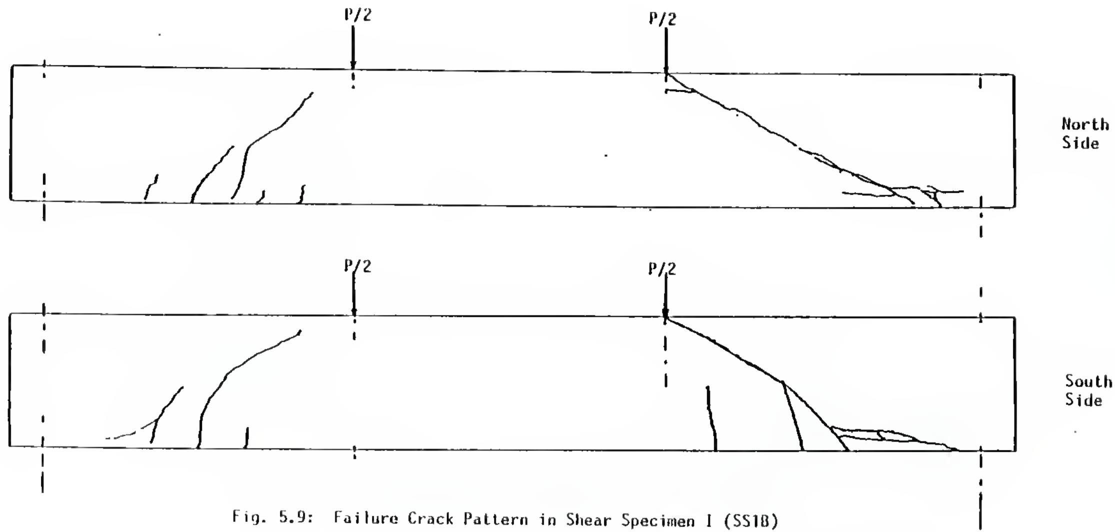


Fig. 5.9: Failure Crack Pattern in Shear Specimen I (SS1B)

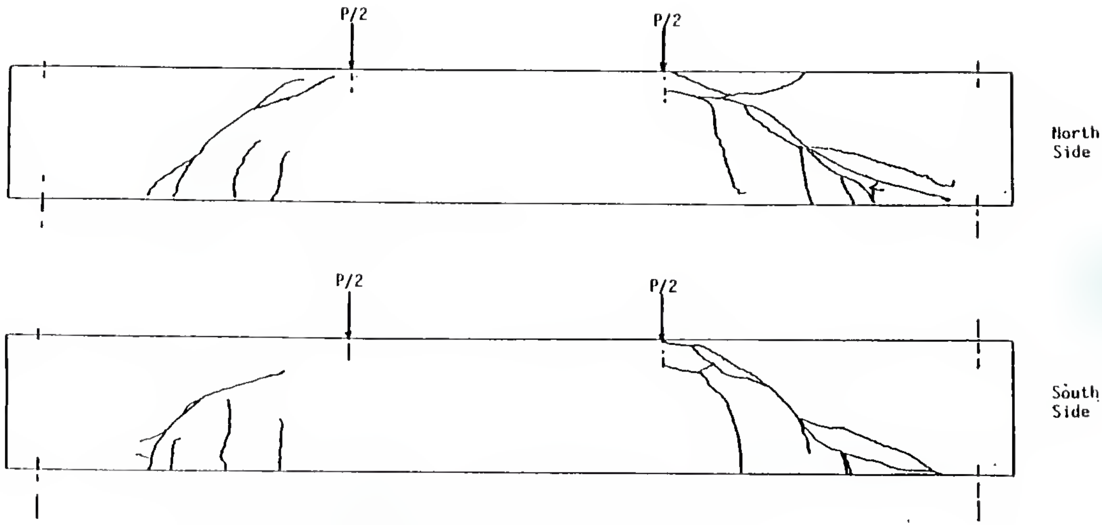


Fig. 5.10: Failure Crack Pattern in Shear Specimen II (SS2B)

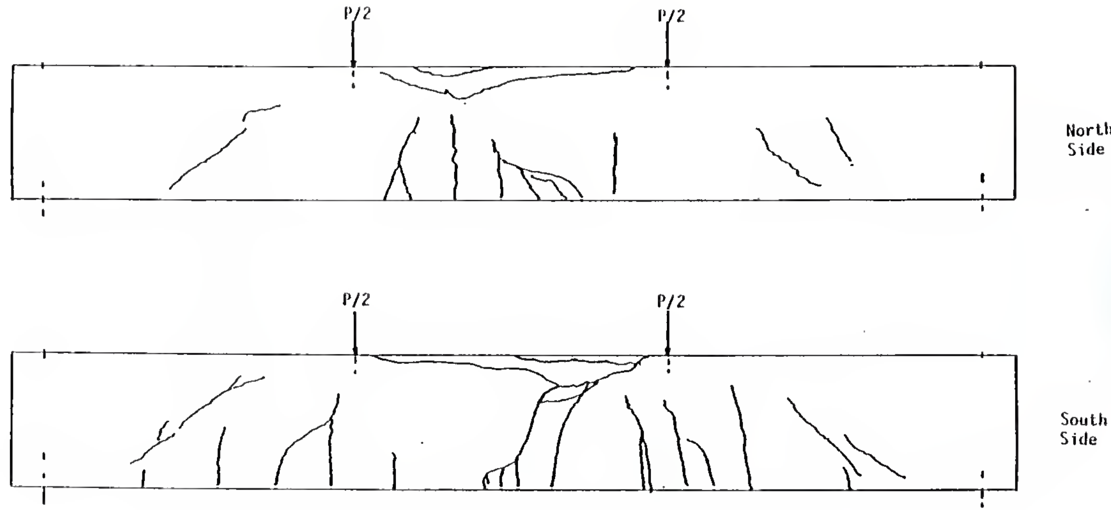


Fig. 5.11: Failure Crack Pattern in Under-reinforced Specimen (UR1)

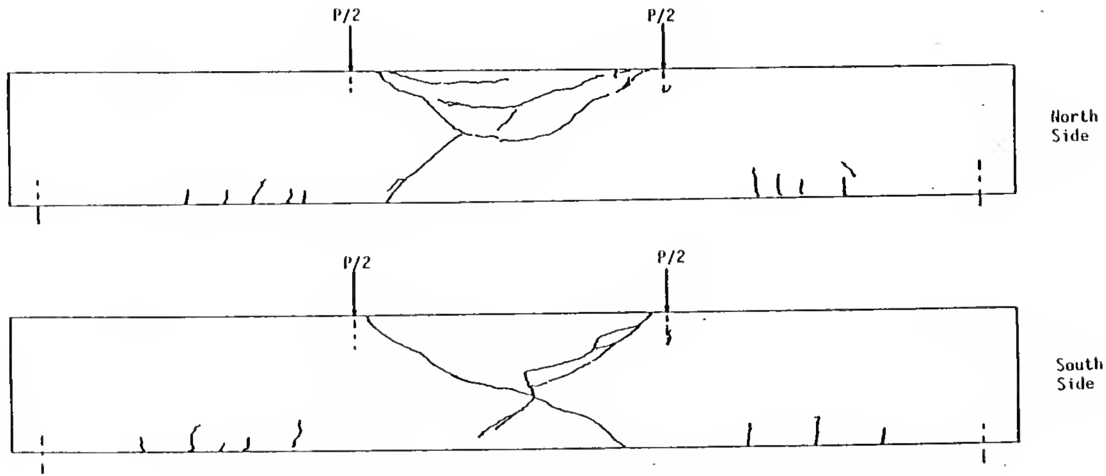


Fig. 5.12: Failure Crack Pattern in Over-reinforced Specimen (OR1)

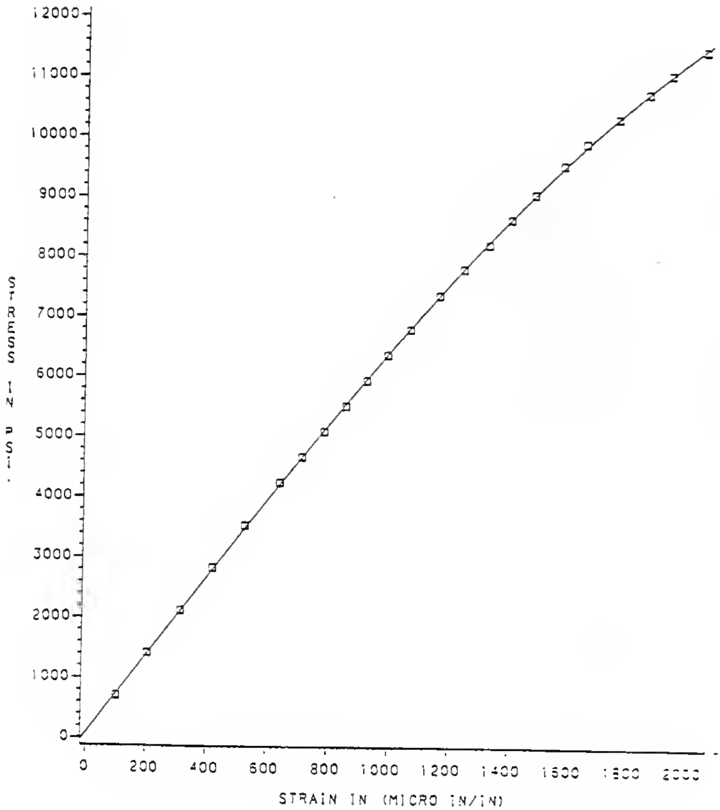


Fig. 5.13: Compressive Stress-Strain for Cylinder 1 (5518)
(f'_c = 11500 psi),(1 in. = 25.4 mm, 1 psi = 6.89 kPa)

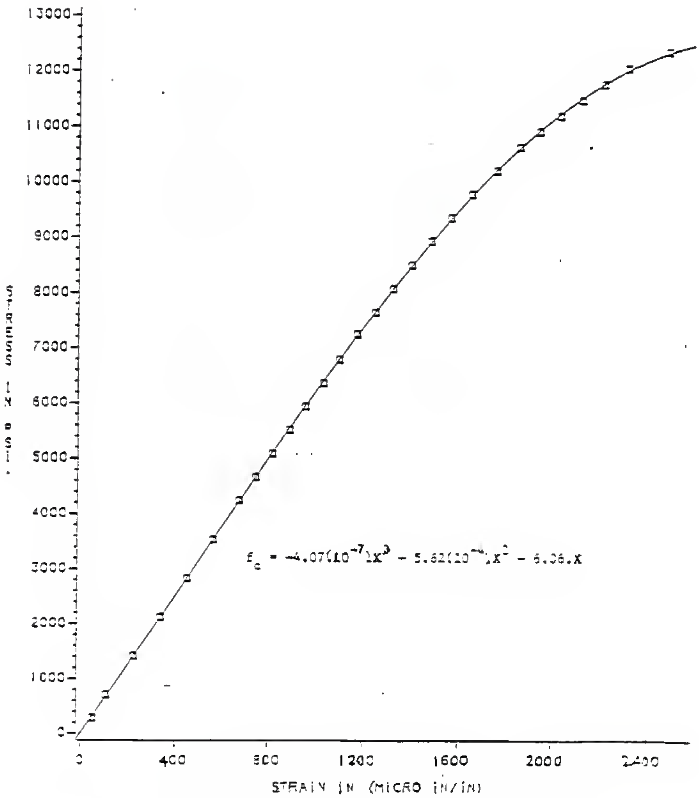


Fig. 5.14: Compressive Stress-Strain for Cylinder 2 (SS2B)
(f'_c = 12310 psi). (1 in. = 25.4 mm, 1 psi = 6.89 kPa)

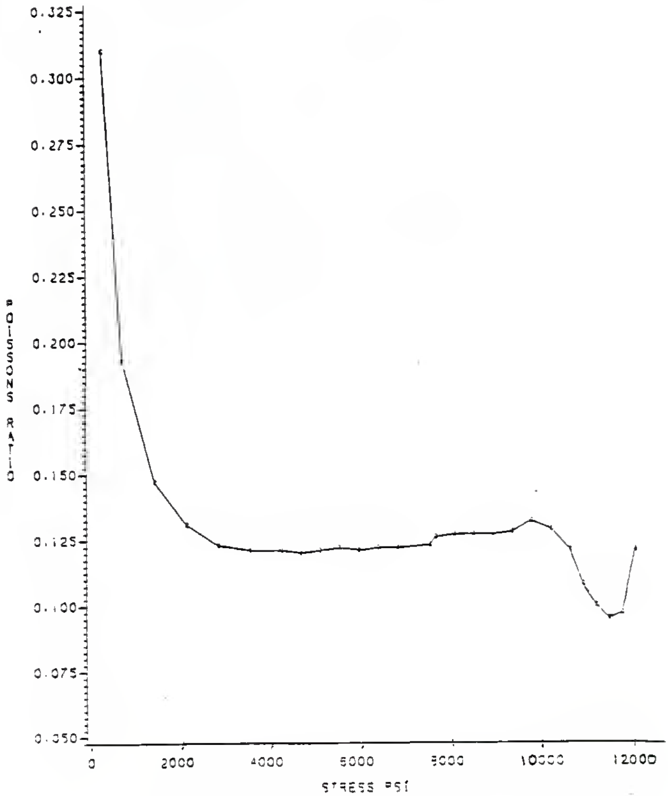


Fig. 5.15: Poisson's Ratio vs. Compressive Stress for Cylinder 2

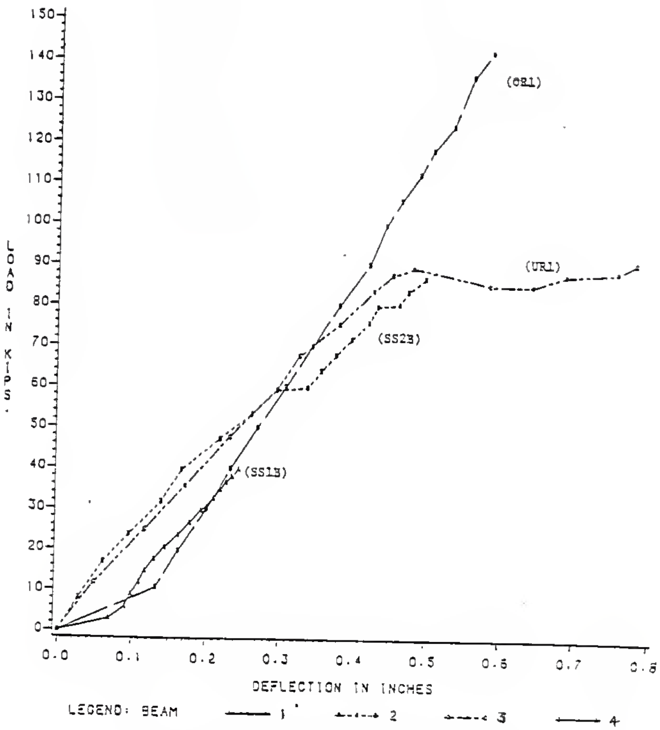


Fig. 5.16: Load vs. Deflection of Test Specimens
(1 in. = 25.4 mm, 1 kip = 4.45 kN)

APPENDIX V

NOTATION

NOTATION

B_1 = coefficient for depth of equivalent rectanglar stress block.

ϕ = strength reduction factor.

p = steel ratio.

p_{max} = maximum steel ratio.

p_{bal} = balanced steel ratio.

ϵ_s = strain in steel.

ϵ_u = strain in concrete.

A_b = area of the parabolic stress block.

A_v = area of the shear reinforcement.

A_s = area of steel reinforcement.

b = width of the beam.

c = depth of neutral axis.

C = total compressive force.

d = effective depth of the beam.

D = total depth of the beam.

E_c = secant modulus of elasticity for concrete.

E_s = modulus of elasticity for steel.

f_c = stress at any distance x from neutral axis.

f'_c = ultimate compressive strength of concrete.

f_s = stress in the steel reinforcement.

f_y = yield stress of steel reinforcement.

l = length of the test beam.

l_{cr} = horizontal crack length.

M_u = actual moment in the beam (calculated using the load).

M_u^2 = triangular stress block moments.

$M_u[\,]$ = rectangular stress block moments.

$M_u§$ = parabolic stress block moments.

N = no. of stirrups.

P = load on the beam.

S = spacing of stirrups.

T = total tensile force in the beam.

V_c = shear force taken by concrete.

V_{cr} = shear stress at inclined cracking.

V_u = ultimate shear force in the beam.

W = unit weight of concrete.

x = distance measured from neutral axis.

$\bar{X}$ = centroidal distance of the stress block from the top of the beam.

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BEHAVIOR OF REINFORCED HIGHER STRENGTH CONCRETE BEAMS IN BENDING AND SHEAR

by

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ABSTRACT

Higher-Strength concrete has been defined as that which has a compressive strength in the range of 8000 to 12000 psi. The purpose of this thesis is to obtain further information on the shape of the compressive stress block at failure for higher-strength concrete beams and to check the validity of the ACI rectangular stress block for higher-strength concrete. The ACI formula for the critical and the ultimate shear was also checked.

Four reinforced beams with nominal strength of 12000 psi were tested in four-point bending. Each beam spanned 7.0 ft. and had the cross-sectional dimensions of 8 in. by 12 in. The beams were reinforced with Grade 60 steel at 0.5 p_b and 1.5 p_b (p_b based on an assumed trianguler stress distribution at failure) and had either no stirrups, or 1/2 the required stirrup area or the full stirrup area. From the strain values obtained, the stress was calculated using the uniaxial stress-strain data. These stress values were used to plot the shape of the stress block and to calculate the maximum moments using triangular, ACI equivalent rectangular, and parabolic stress blocks.

From the results, it was concluded that the shape of the stress block is triangular at low loads and it becomes parabolic-or may remain triangular-at ultimate loads. Therefore the ACI equivalent rectangular stress block should not be used in moment calculations for higher-strength concrete even though it may give a close and conservative estimate of the ultimate moment capacity. The ACI formula for critical and ultimate shear was found to be safe and economical.