STRENGTH AND BEHAVIOR OF THICK WALLED REINFORCED CONCRETE CONDUITS

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# Strength and Behavior of Thick Walled Reinforced Concrete Conduits

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### Abstract
The results of tests of 8 three-opening reinforced concrete conduits designed for soil loadings of 30 k/ft² are presented and discussed. The test specimens were 3/8 scale models of prototype designs, and were constructed with conventional deformed bars and concrete with 1-in. aggregate. The clear span-depth ratios in critical members ranged from 2.5 to 5.6. External loads were applied in both vertical and horizontal directions simultaneously, so that significant thrusts were acting in all members in:

(Cont. on back)
20. addition to the bending and shear forces.

Failure loads ranged from 82.5 to 150 k/ft$^2$ vertical load for members with span-depth ratios of 4.0 or less. Horizontal loads of 60 to 75 k/ft$^2$ produced failures in members with span-depth ratios of 5.6. All members failed in shear.

A lower bound to the test data was suggested as a function of $\sqrt{f_c}$, the span-depth ratio, and the longitudinal compression acting on the member.

No strong influence of the reinforcement ratio was found. When other test results were compared, there did not appear to be a strong influence of the moment-shear ratio as expressed by the variable Vd/M.
TABLE OF CONTENTS

Chapter | page
--- | ---
1. INTRODUCTION | 1
  1.1 General Remarks and Background | 1
  1.2 Object and Scope | 7
  1.3 Administrative Organization and Acknowledgements | 7

2. DESCRIPTION OF TEST SPECIMENS | 9
  2.1 General Description | 9
  2.2 Scaling from Prototype to Model | 10

3. MATERIALS AND CONSTRUCTION | 11
  3.1 Reinforcement | 11
  3.2 Steel Assembly and Placement | 11
  3.3 Concrete | 12
  3.4 Formwork, Casting and Curing | 13

4. LOADING SYSTEM | 15
  4.1 Test Set-Up | 15
  4.2 The Loading System | 16
  4.3 The Hydraulic System | 17

5. INSTRUMENTATION | 19
  5.1 Strain Gage Measurements | 19
  5.2 Load Measurements | 20
  5.3 Dial-Gage Measurements | 21

6. TESTING PROCEDURE | 22

7. BEHAVIOR OF TEST SPECIMENS | 24
  7.1 General Remarks | 24
  7.2 Specimen R4 | 24
  7.3 Specimen R5 | 27
  7.4 Specimen R6 | 28
  7.5 Specimen R7 | 30
  7.6 Specimen R8 | 32

8. DISCUSSION OF TEST RESULTS | 36
  8.1 General Remarks | 36
  8.2 Flexural Strengths of the Test Specimens | 39
  8.3 Shear Strength | 46

9. SUMMARY AND CONCLUSIONS | 57

LIST OF REFERENCES | 59
1. INTRODUCTION

1.1 General Remarks and Background

The "Investigation of Multiple Opening Concrete Conduits" was initiated in 1970 to investigate the strength and behavior and to provide information for the rational design of conduits or "box culverts" suitable for use under earthfill dams and other embankments ranging in depth up to about 250 ft. This report is the third to be issued on the project, and is concerned with the results of tests of the last five of the series of eight specimens tested, plus interpretations of the results of all tests. One earlier report discusses the first three specimens (1)*, and the second (2) the results of an investigation using finite element methods of analysis to predict the behavior of the specimens. The basic properties of the test specimens, the load capacities, and the failure loads are summarized in Table 1.1.

At this fill height, the average earth pressure is about 30 k/ft². The structure must resist this entire load, and in some instances must be capable of resisting more than the overburden since the structure is in effect a very stiff inclusion that may behave as a stress concentration point as the fill compresses. The loading conditions are not similar to those in a tunnel since a tunnel is mined through existing material while the conduit is built first and has the fill placed and very well compacted over it. The techniques of placing soft layers near the conduits which have been used when very deep fills are placed over culverts under roads (3, 4) are

* Numbers in parentheses refer to entries in the List of References.
not applicable since the fill material has to be well compacted in order to properly retain water without endangering the embankment.

The soil mechanics aspects of this problem have not been considered in this investigation. Loadings consistent with current Corps of Engineers practice have been used in the design of the test specimens. Two different loading conditions have been used, and are believed to represent approximately the limiting cases of load distribution. The basic vertical load from overburden considered was 30 k/ft\(^2\). For design purposes, one loading combination consisted of a vertical load of 1.5 times the overburden in combination with a horizontal load of 0.5 times the overburden, giving a ratio of vertical to horizontal pressure of 3:1. The second loading was that of the vertical and horizontal pressures both equal to the overburden. For a conduit such as is shown in cross-section in Fig. 1.1, the first loading controls the design of the horizontal members and the interior vertical members. The second controls the design of the outside vertical members.

It has been assumed that the conduit may be designed by effectively isolating a slice and considering it as a frame with very thick members. This ignores the possibility of axial or bending forces along the longitudinal axis of the structure, which is reasonable as the current practice is to build the structures in ways intended to minimize these forces. Axial tension forces could be induced by spreading of the fill as it settles, but the conduits are built in short sections joined only by waterstops which are not capable of developing significant forces, so the structure moves rather than trying to resist the movements. The same joints minimize the longitudinal bending which might be induced by differential settlements of the material forming the foundation of the conduit.
The applied forces have been assumed to be uniformly distributed on both the horizontal and vertical surfaces of the conduit, and the effects of nonuniform distributions have not been considered.

It may be observed that a circular conduit would generally be a more suitable structure than the rectangular conduits considered in this investigation. However, gate structures are most conveniently fitted to rectangular openings and it may be most economical to maintain a rectangular section rather than provide a transition from rectangular to circular conduits. In addition, if there are requirements for multiple openings, the rectangular configuration may be most suitable.

As an illustration of a case in which the rectangular conduit was most suitable, the cross-section of the conduit which was placed under the Fall Creek Dam, in Oregon, is shown in Fig. 1.1. The central opening provides access to a gate structure located near the center of the dam, and the two outside openings carry water through the dam. The maximum fill over the top of the conduit is about 163 ft. Thinner sections were used for the conduit in the outer parts of the dam where the fill heights were less.

The conduit shown in Fig. 1.1 was designed following provisions similar to those contained in the current Corps of Engineers Engineering Manual, "Conduits, Culverts and Pipes," (5). This guide was prepared assuming fill depths of 60 to 70 ft and maximum earth pressures of no more than 8 to 10 k/ft². In this load range, members with span/depth ratios of 5 or more are adequate. Attempts to use this information for the design of structures supporting 20 k/ft² appears to lead to span/depth ratios of 2 to 3, which is considerably more than is necessary when the results of the current series of tests are taken into consideration.
For example, the horizontal members of the conduit through Fall Creek Dam were 39 in. thick for a 66 in. span, and the design load was 163 ft of fill, which produces an average vertical compression of about 20 k/ft², assuming the structure carries the column of soil directly above it. For contrast, the weakest test specimen in which a horizontal member failed carried 105 k/ft², with a scaled member thickness of 24 in. and a span of 78 in. The strongest carried 150 k/ft², with a scaled member thickness of 36 in. and the same 78 in. span.

Such discrepancies should not be unexpected, however, since the current state of knowledge about the shear strengths of deep members is relatively incomplete. The shear strength provisions of Ref. 5 were based on a study by Diaz de Cossio and Siess (6, 7, 8), and are strictly applicable to members with span/depth ratios of 5 or more. These provisions are not suitable for use when deeper members are necessary because of higher loadings, as they apparently lead to excessive member sizes.

The existing data on the strength and behavior of deep reinforced concrete members were reviewed briefly in Ref. 1. There have been few changes since that time, and the situation still is that most deep beam tests have been conducted on one-span members supported on steel bearing plates resting on steel rollers, the reinforcement has been anchored by welding to bearing plates of some kind, and the beam has been subjected to one or two concentrated loads. Consequently, most of this data is of limited applicability to the design of a multi-span structure supported on monolithic concrete columns which is subjected to uniformly distributed loads. As an additional problem, the members in the conduits considered in this study are subjected to appreciable axial compression forces, and the influence of axial compression
on the shear strength of deep members has not been investigated systematically.

There have been a number of papers concerning deep beams published since Ref. 1 was completed. None of them included information specifically applicable to the conduits problem, but the importance of the supporting conditions are emphasized in Ref. 9. Several of the beams reported were loaded and supported by shear forces distributed over some portion of the depth of the member rather than being supported on the bottom and loaded on the top as has been the case in most tests. Beams with shear loadings and supports did not behave as well and did not exhibit as much ductility as did the conventionally supported members.

An analysis of the strength of deep beams which assumes that one of the modes of failure is crushing of an inclined "rib" located between two parallel inclined cracks is outlined in Ref. 10. This mode of failure has been suggested by others, and the analysis is refined somewhat in this paper. However, it is assumed in the analysis that there are heavy steel bearing plates, and since the plate width enters the strength analysis directly there is no way to use the analyses for this study without extensive modifications.

Lightweight concrete deep beams are considered in Ref. 11. The beams tested were all roller-supported and loaded with two concentrated loads. The effectiveness of shear reinforcement was a major concern of the investigation.

Deep beams are also considered in Ref. 12. The discussion of the behavior is useful in understanding the design problems, but the suggested design stresses are based largely on simple span beams and account for neither continuity nor axial compression forces.
The 1973 ACI-ASCE Joint Committee 426 report (13) on shear in reinforced concrete provides some additional information on the shear strength of deep beams, and in addition explains the background and derivation of the shear provisions for deep beams which are contained in the 1971 ACI Code (14). It appears that an implicit assumption of simply supported members was made when the expressions were developed. The applicability of the Code expressions to the conduit problem has not been established, and the upper limits on average shear stress are only about half the failure shear stresses observed in the tests. This level of disagreement should not be too surprising, though, since some of the test specimens had large axial compressions in addition to the moment and shear force, and the axial forces enhance the shear capacity.

Sec. 11.9.1 of the 1971 Code also contains the phrase "... and the members are loaded at the top or compression face", which casts further doubt as to whether there was any intention that the provisions apply to continuous members. The shear committee report uses different words, but does not really clear up the question.

It must be concluded that very little of the mass of data on the shear strengths of deep beams has any direct relationship to the shear strengths of the conduit members. The differences introduced by continuity, axial forces, different types of bearing devices, and problems of anchorage of reinforcement in most test specimens are serious and not easily accounted for.
1.2 Object and Scope

This report is the final report on the experimental phase of a research program which has extended slightly more than four years. This report is primarily concerned with the presentation of the data from the last five test specimens, of a series of eight, and with the analysis of the data from all specimens.

A later report will summarize specific design recommendations.

Chapters 2 through 6 of this report describe the fabrication, instrumentation, and testing of the specimens. Chapter 7 contains the presentation and a brief discussion of the basic data on strength and behavior of the specimens, including load-deflection and load-strain data. The analysis of the data from all specimens is described in Chapter 8, and Chapter 9 is a summary.

1.3 Administrative Organization and Acknowledgements

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and N. M. Newmark and C. P. Siess, successive heads of the Department of Civil Engineering. Professor W. L. Gamble has provided overall supervision of the project and of the experimental phase of the investigation. Professor Bijan Mohraz, now at Southern Methodist University, directed the analytical phase of the work.

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2. DESCRIPTION OF TEST SPECIMENS

2.1 General Description

The test specimens were designed to represent a "slice" of a multiple-opening reinforced concrete conduit. The opening sizes are scaled from a prototype opening of 6.5 by 9.0 ft. Considerations of the capacity of the loading equipment led to the adoption of specimen dimensions which are 3/8 scale of the prototype dimensions, and a "slice" thickness of 10 in. as shown in Figs. 2.1 and 2.2.

Preliminary approximate calculations of the forces due to 250 ft of overburden, and a literature review of research on deep beam behavior, provided a basis for the selection of the member sizes and the reinforcement ratios. It is current practice with this type of structure to provide 4 in. clear cover over all reinforcement, and the 1.5 in. cover in the test specimens was scaled from this value.

For test specimen R1, the external members were 13.5 in. thick, and the internal members were 9 in. thick, based on prototype dimensions of 3 ft and 2 ft, respectively (see Fig. 2.1). Four #7 deformed bars were used in each member, with two bars at each face, with details as shown in Fig. 2.3 and 2.4.

The dimensions of test specimens R2 through R8 were identical, with internal and external thickness of 9 in., based on a prototype dimension of 2 ft (see Fig. 2.2). Four #6 deformed bars were used in each member of specimens R2 and R3. Specimens R4 through R7 contained four #4 deformed bars each while specimen R8 contained four #3 deformed bars. The bars were placed in each specimen with two bars at each face, as shown in Figs. 2.5 and 2.6.
In accordance with the practice of providing longitudinal crack-control steel on the internal surfaces of conduits, an inner layer of #3 bars was provided, perpendicular to the main reinforcement, in all eight specimens (see Figs. 2.3 and 2.5). This steel was provided in the models mainly because of its possible influence on crack development and had been included in the prototype for control of possible erosion of the concrete by hydraulic forces. The prototype reinforcement was #8 bars at 12 in. spacings.

Table 2.1 lists reinforcement ratios and member thicknesses for all specimens.

Photographs of some of the reinforcement details are shown in Figs. 2.7 to 2.9.

2.2 Scaling from Prototype to Model

The test specimens were accurate geometric scale models of prototype structures. The materials in model and prototype were similar, and consequently introduce no particular concerns of difficulties.

All dimensions of the prototype were multiplied by the scale factor to obtain the model dimensions. All areas are reduced by the scale factor squared, which leads to the same steel ratios in model and prototype. Consideration of the relationships between loads, spans, and section modulus values leads to the fact that uniformly distributed loads in model and prototype must be equal to produce the same stresses, and hence there is no scale factor for uniformly distributed loads.

Consequently, the uniformly distributed failure loads found for the models, as listed in Tables 7.1 to 7.5, are the same as would be expected for prototypes with the same material properties.
3. MATERIALS AND CONSTRUCTION

3.1 Reinforcement

The #7 bars for specimen R1, the #6 bars for specimens R2 and R3, the #4 bars for specimens R4 through R7, the #3 bars for specimen R8 and the #3 bars used to represent the longitudinal steel were all ASTM A-615, Grade 60 material. Typical stress-strain curves are shown in Fig. 3.1, and average values of yield and ultimate stress, and ultimate elongation are given in Table 3.1. All values are substantially above specification minimum values.

3.2 Steel Assembly and Placement

All of the reinforcement was tied into a cage before being placed in the pre-assembled timber form. The #3 bars acted as spacers for positioning the main reinforcement in the members.

At these locations where strain gages were attached, the bar deformations were ground and filed off. A sufficient area was prepared and the strain gage and leads were attached.

The cage was then lowered into the form and further minor adjustments were made to position the reinforcement as accurately as possible. Wooden spacers were used to maintain the correct cover. These were pulled out after the concrete had been placed and partially consolidated.
3.3 **Concrete**

a. **Mix Proportions**

For specimen R1, the mix proportions (by weight) used were 1:4:4 (cement:sand:aggregate) with a water/cement ratio of 0.85. For specimens R2 through R5, the mix proportions were 1:4:4 with a water/cement ratio of 0.9. Specimens R6 through R8 contained mix proportions of 1:4.5:4 with a water/cement ratio of 0.98. Representative stress-strain curves for these concrete mixes are shown in Fig. 3.2.

Type III, high-early strength, cement was used for all specimens. Crushed limestone aggregate, with a maximum size of 1 in., and Wabash River sand were used in the mix. The concrete was mixed in a 1/2 cu yd horizontal pan type mixer.

Twelve full cylinders, 6 by 12 in., and eight half-cylinders, 6 by 6 in., were cast with specimen R1. Nine full cylinders and six half-cylinders were cast with each specimen R2 through R8. The full cylinders were used for compression tests and the half-cylinders were used for split-cylinder tests. All cylinder tests were conducted at the time of the final test of the specimen.

Table 3.2 lists the average compressive strength, average tensile strength from split-cylinder tests, slump, age of the specimen at time of testing, and the initial modulus of elasticity of the concrete.

Dried aggregates were used for all concrete. The aggregate absorbed some of the mix water, resulting in the true water/cement ratio being somewhat lower than nominal values listed above. This is one of the reasons that the concrete strengths were so high for the relatively lean mixes.
used. In addition, casting and curing were under ideal conditions, and the aggregates were very well graded.

3.4 Formwork, Casting and Curing

The test specimens were cast in timber forms made with plastic-coated plywood surfaces. The form was designed so that the member widths in the specimen could be easily adjusted. The member widths of a test specimen correspond to the wall thicknesses in a prototype conduit.

Holes for the rods used to support the specimen during the test were provided by casting in pieces of aluminum tubing through the thickness of the specimen.

The reinforcement cage was placed and positioned in the form using temporary timber pieces wedged between the reinforcement and the sides of the form. The reinforcement cage rested on the ends of the #3 bars. The #3 bars were used to accurately locate the main reinforcement to which they were tied. The temporary timber pieces were withdrawn at the time of casting. Inserts for four lifting hooks were embedded in the concrete before final setting.

Specimen R1 was cast in four batches, with three full cylinders and two half cylinders being cast from each batch. Specimens R2 through R8 were cast in three batches.

Casting was almost continuous with only a short delay of about 10 minutes between the placing of each batch. The concrete was vibrated with an electric internal vibrator. The concrete surface was first smoothed with a wooden screed and then finished with a steel trowel.
About twenty-four hours after casting, the side forms were removed, and the specimen and the control cylinders were covered with wet burlap and plastic sheeting. The wet burlap was removed after seven days. The control cylinders and the specimens were then cured under laboratory conditions until the time of the tests.
4. LOADING SYSTEM

4.1 Test Set-Up

The test specimens, which were three-eighth scale models of a slice of a prototype conduit, were hung in a horizontal position on four steel rods suspended from a supporting frame as shown in the initial progress report, Ref. 1. The vertical and horizontal load directions correspond to the north-south, and east-west directions, respectively.

The soil pressures are simulated by a series of independent loading units. A loading unit consisted of two tie-rods, two steel beam "yokes", and two hydraulic rams. Each loading unit formed a closed loading system (see Fig. 4.1). Consequently, no external reaction system was required, other than the frame supporting the weight of the specimen and the loading units, which rested on the specimen.

Supplementary supports, from the laboratory floor, had to be provided under the hydraulic rams of part of the loading system for specimen R1 to prevent excessive bending of the 3/4 in. diam. tie-rods on which part of the hydraulic rams and beams rested. All tie-rods for the loading system for R2 through R8 were 1 in. diameter and the supplementary supports were required only under the 60-ton hydraulic rams. These supports were necessary only when the system was unloaded.

A photograph of the specimen R1 in the support frame before the loading equipment was assembled is shown in Fig. 4.2, and the same specimen with all equipment in place is shown in Fig. 4.3.
4.2 The Loading System

The uniform soil pressures were approximated by a series of loads applied to the specimen by independent loading units. The system was designed so that for given pressure conditions, the axial load and shear force at the critical sections could be accurately simulated. The bending moments produced approximated those for the ideal uniformly distributed load. The loading units were divided into two main groups: (i) those providing axial load only and (ii) those acting as span loads. A span was considered to be the clear distance between the faces of opposite members. The load on a span was provided by three equally spaced loading units for the short (horizontal) spans, and by four equally spaced loading units on the long (vertical) spans. One loading unit applied axial load directly to each member with the loading units spaced as shown in Figs. 4.4 and 4.5.

The load applied by each loading unit was that required by its "tributary width." As an example, the width for a span unit applying the vertical load to specimen R1, Fig. 4.4, is 9.75 in., while the width for an adjacent unit applying axial load directly to an interior wall is 9.0 in. Consequently, the load on the span unit had to be maintained at about eight percent higher than in the axial unit.

A loading unit consisted of two round high-strength steel tie-rods threaded at each end, two steel beam "yokes", and two hydraulic jacks, as shown in Fig. 4.1. The rods in the north-south direction were 10 ft long, 1 in. in diameter, and were in horizontal planes 6 in. from the centroidal plane of the specimen. The tie-rods in the east-west direction
were made up of two lengths, 3 ft and 12 ft, spliced together with a sleeve nut, and were in planes 9 in. from the centroidal plane of the specimen. For specimen R1, 3/4 in. diameter tie-rods were used in the east-west direction. The 3/4 in. diameter tie-rods were replaced by 1 in. diameter rods for the remaining specimens. All tie-rods had yield stress values in excess of 100 k/in.\(^2\), and most were AISI 4142 heat-treated steel. ASTM A-194 Grade 2H nuts were used on all tie-rods.

The steel "yokes" in the north-south direction were short beams cut from standard 8 by 4 in. rectangular tubing. The "yokes" had holes near each end of both 4 in. faces, through which the tie-rods passed, and internal stiffeners adjacent to the holes.

The steel "yokes" in the east-west direction were made up of two short beam lengths cut from standard 7 by 2 in. channel section, placed back to back. They were held 2.5 in. apart by three special pins, fitted through holes in the channels and secured with nuts at each end. The tie-rods passed through holes drilled through the two outermost pins. The pins at one end of each loading unit acted as a bearing for the tensioned tie rods.

Center-hole rams, mounted on one end of each tie-rod, applied the tension force to the tie-rod. Each rod was equipped with an electrical load-cell. Thirteen loading units were required in the north-south direction, and six loading units were required in the east-west direction.

4.3 The Hydraulic System

The basic parts of the hydraulic system were: Thirty-four 30-ton and four 60-ton, center-hole, double-acting hydraulic rams; three electric pumps; one air-powered pump; and two hand pumps.
The hydraulic system for each specimen was divided into five groups, as indicated in Fig. 4.4. Each group contained either a set of span-loading units only or a set of axially-loading units only. For the test on specimen R1, four groups required one pump each, and the fifth group was further divided between two pumps. The test on specimen R2 required one pump for each of the five groups. Only the three electrical pumps and the air-pressure powered pump were utilized for the test on specimen R3, two of the groups having been connected to the same pump. Tests on specimens R4 through R8 required the three electric pumps and two hand pumps with each pump connected to its own group.

A feature of the hydraulic system was that each ram had an associated load-cell, and could be connected to or disconnected from the hydraulic system by an independent value. Consequently, load could be applied or released in any ram independently of the other rams in the system. Load could also be applied or released simultaneously in all the rams connected to a particular pump.
5. **INSTRUMENTATION**

5.1 **Strain Gage Measurements**

Foil type electrical resistance strain gages, type EA-06-500BH-120 (manufactured by Micro-Measurements), were used to measure the reinforcement strains at the locations shown in Figs. 5.1, 5.2, and 5.3 for the eight test specimens.

Special care had to be taken in mounting the gages on the reinforcement. The bar deformations were first ground or filed off and an area made round and smooth. The surface was thoroughly cleaned with acetone and a metal conditioner, before cementing the gage and a lead-tab to the rebar with Eastman 910 adhesive. The lead-tab and the gage were originally mounted on backing tape. When the adhesive had matured, the backing tape was removed with the aid of acetone. The electrical leads were then soldered to the lead-tab and the gage. The leads were secured to the bar with tape, and a layer of waterproofing was carefully applied over the gage area. The waterproofing also provided protection against damage to the gage during the casting of the specimen. Waterproofing was accomplished by applying a piece of an 1/8-in. sheet of Schtch No. 2200 E-Z Seal electrical insulation over the gage area, and pressing firmly to the bar. The material is a semi-cured neoprene compound.

Paper backed electrical resistance strain gages, type A-12 (manufactured by BLH Electronics, Inc.), were used to measure strains on the concrete surface at the locations shown in Fig. 5.4 for specimens R6 and R8 and in Fig. 5.5 for specimens R5 and R7.
Care also had to be taken in mounting the gages on the concrete surface. Because of the nature of the location (the corners of the specimen had the roughest surface because the form at that point was made up of many plywood ends), the surface of the specimen had to be ground excessively and then sealed with "Hydrocal" plaster. The prepared surface was then cleaned with acetone. After surface preparation was completed, the gage was cemented to the concrete surface. The electrical leads were then soldered directly to the gage.

The strain measurements were monitored on a 100-channel Pivan strain gage indicator panel. One strain gage reading can be visually displayed on the Pivan panel at any particular time. The strain gage readings were automatically displayed consecutively and recorded on an attached teleprinter at the rate of about one reading per second. The teleprinter provided an immediate print-out of the strain gage readings and also recorded the data on punched paper tape. The punched-tape records were later converted into IBM punched cards which were used as input data for a computer program. The computer program compiled and reduced the data and provided a print-out of the strain measurements at each location at each specific load level.

5.2 Load Measurements

A total of 38 load cells were used to measure the applied loads. The load cells were axially loaded thick-walled cylinders machined from 6061-T651 aluminum rods. Each load cell was 6 in. long with an outside diameter of 2 in. and an inside diameter of 1 1/8 in. The four strain gages mounted on the external surface of each load cell were wired to form a four-arm bridge measuring circuit.
The load cell capacity was computed to be 57 kips at a stress of 30 kips. The load cells were calibrated to a maximum load of 50 kips. Twenty-eight load cells, which had been used in an earlier study, had sensitivities between 80 and 87 lb per dial division (10^{-5} strain). The ten new load cells had sensitivities between 78 and 80 lbs per dial division. Before the tests on specimens R2 to R8 load cells were chosen at random and re-calibrated. The sensitivities showed variations of less than 0.5 percent.

As with the strain gages, the load cells were monitored on the Pivan strain system, load cell readings could be automatically displayed, and were recorded by the teleprinter. In this way immediate inspection of the load state could be carried out at any particular time, and permanent records made as desired. The computer program converted the load cell readings to applied loads, gave average load values for the rams in each group and provided a comparison between the desired and the actual loadings.

5.3 Dial-Gage Measurements

The relative displacement at mid-span of each member was recorded by dial gages. Light metal frames were erected off the center of both of the interior members and the gages were mounted on the metal frame and located as shown in Fig. 5.6.

The deflections reported are changes in height or width of frame openings due to all deformations, including shortening of the "columns" and, in the case of the vertical deformations, were obtained by adding the deflections from two different gages, such as 2 plus 3 or 7 plus 8.
6. TESTING PROCEDURE

The test procedure used on specimens R1, R2, and R3 described in the first progress report (1) differs slightly from that used on specimens R4 through R8. That testing procedure first used consisted of initially loading the specimen to 30 k/ft$^2$ or 25 k/ft$^2$ for R1, R2, and R3 in 5 k/ft$^2$ increments. After each 5 k/ft$^2$ loading was placed, readings were taken and a careful examination of the instrumentation and specimen were made. The hydraulic system was closely watched during this time. After the 30 k/ft$^2$ load was reached and readings taken, the load was removed. This constituted the preliminary check-out.

The full-scale test consisted of two 15 k/ft$^2$ increments followed by 7.5 k/ft$^2$ increments until failure. After each loading a set of readings were taken and a careful examination of the specimen was made.

Using the knowledge gained on the first three tests a much simpler test procedure was formed for specimens R4 through R8. An initial load of 5 k/ft$^2$ was applied uniformly around the specimen. All hydraulic apparatus and instrumentation were then checked out. All specimens were then loaded to 15 k/ft$^2$ and then to 30 k/ft$^2$. From there the load was increased in 7.5 k/ft$^2$ increments to failure, except for some deviations for specimens R7 and R8 as detailed in Chapter 7.

The loading units were divided into two main groups: (i) those providing axial load only (3 groups) and (ii) those acting as span loads (2 groups). A further subdivision separated those acting vertically from those acting horizontally (see Fig. 4.4). The loading condition for
specimens R4, R6 and R8 where the vertical load was three times that of the horizontal load, indicated that the horizontal members were the most critically stressed in shear. For this loading condition the horizontal loads were applied first, then the loads bearing directly on columns, and finally the loads on the horizontal spans.

In specimens R5 and R7, where the end vertical members were most highly stressed in shear, the vertical loads were applied first, followed by the horizontal loads bearing directly on the horizontal members. The horizontal loads acting on the end vertical members were applied last.
7. BEHAVIOR OF TEST SPECIMENS

7.1 General Remarks

The overall observed behavior of the test specimens is described in the following sections. Photographs of the failed specimens, load-strain curves for various points on the reinforcement, and load-deflection curves are presented. The progression of cracking with increased load is described, and comments on various details, important and otherwise, are made in an attempt to increase the understanding of the behavior of the last five test specimens in the series, R4 through R8. The corresponding information on the first three specimens, R1 to R3, is presented in Ref. 1.

Most of the interpretation of the test data is reserved for Chapter 8 of this report, where all eight specimens are discussed.

7.2 Specimen R4

7.2.1 General Remarks

Specimen R4 was tested at a loading ratio of 3:1 (vertical to horizontal) until a load level of 120 k/ft\(^2\) was reached. At this point, only loads bearing on the horizontal span were increased until the specimen failed in shear at approximately 132.5 k/ft\(^2\). A detailed description of the test follows and photographs of the specimen are shown in Figs. 7.1 and 7.2.

7.2.2 Full Scale Test

The loading pattern used during the test is tabulated in Table 7.1. In what became a standard procedure, the specimen was first loaded to
5 k/ft$^2$ in both vertical and horizontal directions to test the hydraulic and recording systems. Next, the vertical load was raised to 15 k/ft$^2$ and then to 30 k/ft$^2$. Twelve further increments of 7.5 k/ft$^2$ each raised the load to 120 k/ft$^2$. Although the specimen was still intact, the nature of the cracks suggested that failure was imminent. Therefore, it was decided to lock the horizontal jacks against further increases in hydraulic pressure and to increase the pressure slightly in the span loading vertical jacks. This had the effect of increasing the horizontal as well as the vertical loads since the horizontal loading units frustrate the tendency of the outer vertical spans to bulge outwards when only the horizontal spans are loaded. It was necessary to apply three small load increments before the specimen failed in shear in an outer horizontal span. The vertical load at failure was approximately 132.5 k/ft$^2$, based on an analysis of the load cell readings after the test and load cell readings made at the time of failure.

Flexural cracking in the positive and negative moment zones commenced at 30 k/ft$^2$ and was confined mostly to the horizontal spans throughout the test. Shear cracking, which commenced at 82.5 k/ft$^2$, was also confined to the horizontal spans. It was the severity of a large shear crack in an outer horizontal span which prompted the change in loading process when 120 k/ft$^2$ was reached.

The specimen failed in shear in an outer horizontal span (Fig. 7.2). The two principle failure planes are of the shear-diagonal tension variety. Both planes are poorly defined in location because of the extensive sundering of the adjacent concrete. The fact that most of the concrete surrounding
the outer reinforcement in the failure zone was stripped away suggests that this failure is most properly classified as a shear-compression failure.

The strain gages (Figs. 7.3 - 7.11) behaved in a manner that proved to be typical of all the 3:1 specimens. Initially, all gages behaved in a manner consistent with the nominal positive and negative moment zones, with due allowance for axial loads. However, at loads ranging from 37.5 k/ft² to 60 k/ft² (in R4), nominally compressive gages in the negative moment regions of horizontal members went tensile. The reason for this appears to be arch formation which has been observed in a previous test specimen (R2). The onset of the tensile strains was often accompanied by a sharp decrease in slope of the load-strain curve (Figs. 7.3, 7.5, 7.6). The maximum strains reached approximately the yield strain of about 0.0024 in a number of locations, but never greatly exceeded this value.

The reversal of sign of strains in many gages in nominally compression regions may be a sign of the complete loss of bond which can accompany the development of an equivalent arch in a deep beam. Gage 2, Fig. 7.3, is a good example of this behavior. In this case, the extension of the bar into the joint area was adequate to allow the development of about 60 k/in.² stress before the specimen failed.

The strains in the vertical members indicated that the interior members were subjected to nearly axial compression, with only very small bending components. The midheight section of the exterior vertical members also had very little bending moment, as is consistent with the analytical findings reported in Ref. 2, while the end sections had appreciable moments acting.
The dial gage readings (Figs. 7.12 - 7.17), also exhibited what was to become a standard response pattern for the 3:1 loading specimens. Generally, opposing horizontal spans moved toward one another while the vertical spans bulge outward. It should be noted that load-deflection and load-strain curves generally flattened after a brief period of linear behavior. Sharp reductions in stiffness often occurred at the initiation of flexural cracking, with subsequent continued but more gradual reductions.

7.3 Specimen R5

7.3.1 General Remarks

Specimen R5 was tested with a constant loading ratio of 1:1 (vertical to horizontal loads). Failure occurred at a load of 75 k/ft² and no further testing was carried out. A detailed description of the test follows and photographs of the conduit are shown in Figs. 7.18 and 7.19.

7.3.2 Full Scale Test

The loading pattern used for Specimen R5 is tabulated in Table 7.2. A load level of 5 k/ft² was set initially and then raised to 15 k/ft² and, then to 30 k/ft². Successive increments of 7.5 k/ft² were applied. When the load of 75 k/ft² was being applied, a sudden shear failure occurred in a vertical side member. All loads were then removed and the test concluded. Until the failure occurred, all visible cracking behavior was flexural in nature. Positive moment cracking first appeared at a load level of 45 k/ft² and was confined mostly to the two vertical side members. Negative moment cracking developed in the two vertical side members at 45 k/ft² and in the horizontal members at 52.5 k/ft². The failure occurred at both ends of a vertical side member (Fig. 7.19). Both failure planes were inclined across the member from the inner fillet at an angle of approximately 45 degrees.
The strain gage measurements (Figs. 7.20 to 7.28) indicate mostly compressive stresses in the horizontal members up to failure. The sole exception is an outer gage in the negative moment region adjacent to a vertical side member (Fig. 7.20). Here, the horizontal side force was large enough to cause rotation of the joint but it could not overcome the flexural tensile stresses.

In the exterior vertical members, the inner gages in the negative moment regions (Fig. 7.25) registered compressive strains throughout the test so it would appear that arching action and loss of bond did not play a major role in the failure. No strains approached the yield strain before the shear failure occurred.

The dial gage measurements, (Figs. 7.29 to 7.34) indicate that opposite members always moved toward one another. The vertical side members, which generally bulged outwards in the 3:1 conduits, were restrained from going so here by the tripled horizontal load. The horizontal deflections in the two end openings were about the same, up to failure. There was a marked reduction in stiffness at 67.5 k/ft². This change can also be noted in the steel strain measurements, Figs. 7.25 and 7.26, but is less pronounced there.

7.4 Specimen R6

7.4.1 General Remarks

Specimen R6 was tested at a constant loading ratio of 3:1 (vertical to horizontal). The specimen failed in shear at a load of 105 k/ft². A description of the test follows and photographs of the conduit are shown in Figs. 7.35 and 7.36.
7.4.2 Full Scale Test

The loading pattern used during the test is tabulated in Table 7.3. The specimen was first loaded to 5 k/ft². Load levels of 15 k/ft² and 30 k/ft² were then followed by ten successive increments of 7.5 k/ft² each. As the load was being raised to 105 k/ft², a sudden shear failure occurred in an outer horizontal span. All loads were removed and no further testing was carried out on this specimen. Virtually all cracking, both shear and flexural, was confined to the horizontal members. Flexural cracking began at 37.5 k/ft² in the negative moment zones and shear cracking began at 67.5 k/ft² in the interior end of a horizontal side span. Failure was of the shear (diagonal tension) type. There was extensive spalling on either side of the two failure planes, particularly at the exterior negative moment section. Also, this failure plane ruptured the fillet but at the interior negative moment section the failure plane was an extension of the fillet line (Fig. 7.36).

Strain gages (Figs. 7.37 to 7.45) in the positive and negative moment regions of the vertical members behaved in a manner consistent with the positive and negative moment zones in a frame. This is to be expected because the vertical members had little shear cracking, hence the "arch" was not allowed to form. Positive moment gages in the horizontal members also exhibited behavior consistent with the expected strain pattern. However, nominally compression gages in the negative moment zones of the horizontal spans began registering tensile strains at loads ranging from 37.5 k/ft² to 82.5 k/ft². The onset of tensile strains at these locations has been observed in other tests (see Secs. 7.2 and 7.6, and Ref. 1) and is apparently the result of arching action. The effect of arch formation on the stiffness
of the structure can be seen in the sharp decrease in the slope of the load-strain curves at the commencement of tensile readings (Figs. 7.37 and 7.38).

The great irregularities which appear at loads of 37.5 and 60 k/ft² in Fig. 7.38 are apparently due to a switch in the strain recording equipment not making contact, as the reading of zero strain is consistent with an open circuit and not with the behavior of the structure.

The dial gage data is shown in Figs. 7.46 to 7.51. As is characteristic of the 3:1 (vertical to horizontal) specimen, the vertical side members bulged outwards (after an initial contraction) and opposing horizontal spans moved toward one another. The middle horizontal span was somewhat stiffer than the end spans but all load-deflection curves flattened as the load increased. The horizontal deflections of the end openings were rather irregular with increasing load. This may be the result of minor deviations of the horizontal loads from the desired values of 1/3 the vertical load.

7.5 Specimen R7

Specimen R7 was tested at an initial loading ratio of 1:1. Following the failure of a vertical side member at 60 k/ft², all horizontal span loads were reduced to zero. Vertical loads were then increased on the intact portions of the specimen until a second failure occurred in a horizontal span at 82.5 k/ft².

A description of the test follows and photographs of the specimen are shown in Figs. 7.52 to 7.54.

7.5.1 Full Scale Test

The loading pattern used during the test is shown in Table 7.4. The first load increment reached 5 k/ft² in both directions and was intended
primarily as a test of the hydraulic and recording systems. The load was then raised to 15 k/ft², and then to 30 k/ft². Four successive increments of 7.5 k/ft² each followed. Just after the load was raised to 60 k/ft², a sudden shear failure occurred in a vertical side span. The horizontal and vertical loads bearing on the failed span were reduced to zero. The horizontal axial loads and the vertical loads bearing on the intact conduit sections were further increased in increments of 7.5 k/ft². A shear failure occurred in a central horizontal span just after a load of 82.5 k/ft² was achieved.

In the initial test, cracking was flexural up to failure and was confined principally to the vertical members. Positive moment cracking first appeared at 30 k/ft² and negative moment cracking commenced at 37.5 k/ft². Failure occurred at 60 k/ft². The failure plane constituted an extension of the line of the fillet and flattened slightly as it headed into the span. No other shear cracks were observed in the initial test.

All load-strain and load-deflection curves have major discontinuities at 60 k/ft² vertical load, as a result of the failure of the end member followed by removal of most of the horizontal loads.

The strain gage readings (Figs. 7.55 to 7.63) closely followed the pattern set in the previous 1:1 specimen tests (R3, R5); i.e., strains had signs consistent with anticipated regions of positive and negative moment with due allowance for axial loads. In most locations, strains responded linearly with load until a load of 52.5 k/ft² was reached. After that, the strains increased at much greater rates. The dial gage readings (Fig. 7.64 to 7.69) were also consistent with previous 1:1 test results. Opposing spans moved toward one another in a generally linear manner up to initial failure.
After the first failure, the loading pattern was modified as described earlier. The loads were then increased incrementally until a second shear failure at 82.5 k/ft². As the load was increased extensive shear cracking occurred in most of the horizontal members. The initial response of the strain gages to the new loading system was a sharp increase in tensile strains in most locations in the horizontal members. Some evidence of arching action is to be found in the tensile reading in the nominally compression gage in the negative moment region of the central horizontal member.

The second failure plane also started at the inner edge of the fillet and sliced inward to the outside surface of the member, closely following the 45° fillet line. This shear failure was along a single, well defined plane with no significant crushing of adjacent material.

7.6 Specimen R8

7.6.1 General Remarks

Specimen R8 was the last conduit to be tested. It was tested at a loading ratio of 3:1 until an initial failure occurred at 105 k/ft² applied vertical load. An attempt to continue testing other spans of the specimen was aborted when a second failure occurred while the load was being raised back to its initial failure level, probably at a load of about 90 k/ft². Photographs of the specimen are shown in Figs. 7.70 to 7.72.

7.6.2 Full Scale Test

The loading pattern used for specimen R8 is tabulated in Table 7.5. An initial load of 5 k/ft² was applied to check the testing apparatus. Load levels of 15 k/ft² and 30 k/ft² came next, which were then followed by ten successive increments of 7.5 k/ft². Just after the load reached 105 k/ft², a shear failure occurred in an outer horizontal span. As a result of the
failure, considerable load and oil pressure were lost. As the pressure was being raised to continue testing on the intact portions of the specimen, a shear failure occurred in a horizontal end span different from the one where the first failure occurred. Following the final readings, the loads were removed and the test was concluded.

Flexural cracking was confined mostly to the horizontal spans throughout the test and appeared in the positive moment regions at 37.5 k/ft$^2$ and in the negative moment regions at 52.5 k/ft$^2$. Generally, most flexural cracks penetrated further with increasing load.

Shear cracking was confined principally to the horizontal end spans and first appeared at a load of 82.5 k/ft$^2$. As with the flexural cracking, most cracks penetrated further with increasing load.

Initial failure occurred just after a load of 105 k/ft$^2$ was applied. The outer failure plane sliced through the fillet and was slightly inclined there but became almost vertical as it approached the outer surface. It appears that this is the result of a pure shear failure - in contrast to all previous shear failures which have been diagonal tension failures. The inner failure plane was a diagonal tension failure since it sliced inward from the fillet at an angle of about 45°. The shear failure at the exterior negative moment section probably occurred immediately after the failure at the interior negative moment section. The steep inclination of the crack may be a result primarily of flexural deformation followed by a shear failure when the effective area of concrete had been greatly reduced.

Following the initial failure, all horizontal loads and the vertical loads bearing on the failed span were removed. As a result of the failure, the specimen contracted vertically considerably and it was necessary to raise the vertical loads up to their pre-failure levels in order to resume
testing. The load was still below the 105 k/ft\(^2\) (probably about 90 k/ft\(^2\)) when a shear failure occurred in the other horizontal end span of the same member. The principal failure plane was located on the inner end of the member and extended inward from the fillet at an angle of about 45°. At the top of the plane, some concrete was broken off exposing the reinforcement. Also, there was a large subsidiary crack which started at the top of the failure plane and wound its way down to the fillet, at times running parallel to the reinforcement. Thus, this second failure can best be classified as a shear-compression failure, with bond splitting also playing a major role.

The strain gage readings, (Figs. 7.73 to 7.81) closely follow the pattern set by the previous 3:1 specimens. The readings are consistent with the nominal areas of positive and negative moment except for the compression gages in the negative moment areas of the horizontal members. In these gages, tensile strains, indicating the presence of arching action, begin at loads ranging from 60 k/ft\(^2\) to 90 k/ft\(^2\). The general strain gage response can be characterized as having an initial linear region followed by progressive flattening of the curve. Some of the negative moment gages indicate major disruptions were occurring by the time a load of 97.5 k/ft\(^2\) was reached.

Likewise, the dial gage readings (Figs. 7.82 to 7.87) also resemble those of the previous 3:1 specimens. The vertical side spans bulged outwards while opposing horizontal spans moved toward one another. As was the case with the strain gage readings, the response curves gradually flattened with increasing load.

The test of specimen R8 was plagued with oil leaks in the hydraulic equipment, and the testing was stopped several times to replace seals and hoses, and to retighten fittings. In some instances it was necessary to
unload rams completely, but when this was done they were unloaded in pairs, repaired, and reloaded with little apparent effect on the structure.
8. DISCUSSION OF TEST RESULTS

8.1 General Remarks

This chapter is basically divided between discussions of the flexural strengths, Sec. 8.2, and the shear strengths, Sec. 8.3, of the members. In most outward appearances the failures were shear failures, but in some cases it appears that the flexural forces may have had a significant influence on the overall behavior of the specimens.

The strengths of the conduit specimens can be reasonably evaluated only after some discussion of the details of the applied loads. The conduits were designed for uniformly distributed loads. The locations and magnitudes of the jack loads were selected so that they applied exactly the same total force as would have been obtained from the ideal uniformly distributed loading case. The sum of the shears at the two ends of any member, considering sections at the faces of the supporting columns, is the same for either the ideal or the actual loading situation.

However, on the basis of cutting conventional freebodies, the shear force in the test specimen remains constant in the region between the face of the support and the first load, while the shear force diminishes continuously in the same region if the load is uniformly distributed.

It may be argued that, in members of the proportions of those used in the conduit specimens, the distribution of the surface loads is not the only factor to be considered. The distribution of the loads on the plane at mid-depth of the member may give a more reasonable evaluation of the effective distribution. The loads were spaced at about the member depth, and covered
about 35 percent of the surface area (the loading beams were 4 in. wide, but had rounded edges so the contact area was about 3.5 in. wide). If the load is assumed to spread at a 45 degree slope within the depth of the member, there is considerable overlap of the areas of influence of adjacent loads by the time mid-depth is reached.

The finite element program described in Ref. 2 was used to investigate the differences between uniformly distributed loads and equivalent concentrated loads. Some of the results are shown in Figs. 8.1 to 8.5. The stress values are for an elastic analysis, with vertical loads of 3 k/ft² and horizontal loads of 1 k/ft², and are node-point stresses. For the uniformly distributed loading case, loads were applied at every surface node point. Each concentrated load was replaced by two concentrated loads, spaced 2.5 or 4.0 in. apart to simulate the effects of having the actual loads distributed over a finite area, located as shown in the figures.

The vertical stresses acting on three horizontal planes at different levels in the member are shown in Fig. 8.1. As would be expected, the closer the plane is to the loaded face, the greater the differences in stresses between sections directly under the loads and sections between loads. The stress distribution never becomes uniform, but the variations are very much reduced at sections B and C, as compared with those at section A, 1.5 in. from the loaded surface. The high stresses at the ends of the member for section C are a result of the vertical reactions at the supporting columns.

The distributions of shear stresses along plane C are shown in Fig. 8.2 for both the concentrated loading case and the uniformly distributed loading case. The two distributions are very similar, with both having the same trends.
The shear stress distributions on plane B are also similar for the two loadings, though there are somewhat greater differences between the two.

The horizontal stresses at the two negative and midspan positive moment sections of the end span horizontal member are shown in Fig. 8.3. It is clear that on the basis of the finite element solutions there are only very small differences between the two loading cases, and that the distributed loading has been reasonably approximated by the use of the equivalent concentrated loads.

On the basis of the generally good agreement between the two load distributions, the remainder of the analysis of the results of the tests has been conducted assuming that the concentrated loads actually applied to the test specimens were exactly equivalent to the ideal uniformly distributed loads. These uniform loads will be considered when computing shear forces on various potential critical sections.

In addition to the results of the finite element solutions, the appearance of the inclined cracks in the test specimens also supports the assumption. In nearly all tests of deep beams under one or two concentrated loads, the major inclined cracks connect the inside of the bearing and the nearest load point. In the conduit specimens, the major inclined cracks run from approximately the tip of the fillet to the edge of the central load, going below the first concentrated load. This is approximately the path the crack would be expected to take if the load were uniformly distributed, and is certainly not the path it would take if the first concentrated load were a controlling element in the behavior.

The distributions of horizontal stresses on sections at midspan and at the tip of the fillet in the interior span are shown in Fig. 8.4. Only stresses due to the concentrated loading are shown.
The distributions of vertical stresses on sections at midheight and at the tip of the fillets near the end of a vertical member are shown in Fig. 8.5. These stresses were produced by a load of 3 k/ft\(^2\) acting in both the horizontal and vertical directions, with the load assumed to be uniformly distributed in the finite element idealization.

8.2 Flexural Strengths of the Test Specimens

Since all members were subjected to some combination of moment and thrust, in addition to the shear forces, moment-thrust interaction diagrams were prepared for the exterior members of each specimen. The diagrams are shown in Figs. 8.6 through 8.10, and where possible pairs of specimens have been shown in the same graph. The diagrams themselves require comment, as the shapes are somewhat different than the classical shapes given in standard text books. These diagrams have much more curvature above the balance point thrust than is normal. This is a result of two factors working in combination. First, the cover over the steel at the compression face is relatively large, which leads to low strains and stresses in that steel, and secondly, the compression steel was quite strong, with yield stresses of 70 k/in.\(^2\) and higher. The strong steel requires high strains to develop its yield stress, and the yield stresses simply are not developed in these members unless the axial loads are very large and the bending moments relatively small.

There is no well defined balance point for R8, Fig. 8.10. The steel ratio was very low for this specimen, and the discontinuity in the curve accompanying yielding of the tension steel simply is not evident at the scale used for the graph. The shape is approximately the same as for an unreinforced section, except that there is some moment capacity when thrust equals zero.
The curves for a prototype structure would also have the same characteristics if Grade 60 steel were used, since the relative proportions of the sections would be the same. Both the absolute cover over the compression steel and its effective depth relative to the effective depth of the tension steel are significant.

The interaction diagrams were computed following the provisions of the 1971 ACI Building Code (14), and the equivalent rectangular stress block was used when finding the concrete compression force. No correction was made for the area of concrete displaced by the compression steel except when the force for a pure compression failure was being computed. The curves were computed point-wise, starting from a series of assumed strain distributions, one of which is illustrated in Fig. 8.11. The limiting compression strain was taken as 0.003 for the concrete, and no intermediate points were computed for axial loads higher than that causing the limiting strain of 0.003 at one face of the member and zero strain at the other.

Once the interaction diagrams had been established, the next step was to determine whether the combination of moment and thrust imposed on particular members is on or inside of the line representing failure. In general, it is not possible to be very exact in this process because the distribution of moments, between positive and negative sections, is not known very precisely. However, definite limits can be established.

The minimum possible moments occur when the two negative moments at the ends of the member are both equal to the positive moment. No other statically admissable bending moment diagram results in lower moments, which may be expressed as
where \( w \) = load per unit length, and \\
\( l \) = span.

The span was taken as the clear span, tip to tip of the fillets, in the evaluations using this equation. This was done after some trials using the span face to face of supporting members and after observing that the failure sections were nearly always at the tip of the fillets rather than closer to the support face.

The axial forces were of course known precisely for the horizontal members. They had to be estimated for the end vertical members since the end moments were neither symmetrical nor precisely known in the end spans, and this was done using an elastic analysis run on the Strudl Computer Program. In this analysis, it was assumed that the members were rigid in the area between the tip of the fillet and the centerline of the supporting member, a length of 6.75 in. at each end of each member.

It was also assumed in the evaluation of the flexural problem that the maximum relevant moment is the negative moment at the tip of the fillet, as determined by the same elastic analysis.

The use of the moments and thrusts in determining whether a flexural failure was possible can be illustrated by reference to Fig. 8.7. The two straight lines sloping up to the right from the origin represent the load paths from the elastic analyses of specimens R-2 and R-3. The path for R-3, which was subjected to equal horizontal and vertical loads and suffered the failure of an end vertical member, terminates at a thrust of 125 kips and a moment of about 430 k-in., corresponding to the measured failure load of 67.5 k/ft². The point is obviously well inside the failure curve, and it is
possible to rule out the possibility of the failure being primarily initiated by flexural stresses.

The elastic load path for R-2, for which vertical loads were three times the horizontal loads, terminates at an axial thrust of 71 kips, and a moment of 430 k-in. The horizontal line through this point represents the thrust acting when the section failed, and is a known value. The vertical line at the left end of the horizontal line represents the minimum possible bending moment required for equilibrium, as determined from Eq. 8.1. Thus, the combination of 71 kips thrust plus 280 k-in. moment represents a limiting condition at failure. The maximum moment in the span may have been larger, but it cannot have been smaller, and the thrust is a known quantity. The elastic moment represents a reasonable maximum moment.

The same general information about the other test specimens is shown in Figs. 8.6, and 8.8 to 8.10. Specimens R-5 and R-7, which had the same vertical and horizontal loads, also apparently failed without major influence of flexure, and the elastic load paths end at points well within the interaction surfaces.

However, for specimens R-4, R-6, and R-8, the elastic load path lines end outside of the interaction surfaces. In each case the minimum moment points, which correspond to full redistribution of the moments to equalize positive and negative moments, are well inside the interaction surface, indicating that flexural equilibrium could exist.

In specimens R4, R6, and R8, the measured steel strains in the end span positive and interior negative moment sections generally reached approximately the yield strain by the time of the shear failure at the end of the test. In no case except the positive moments in R8 did the strains exceed the yield
strain by any significant margin, and in that specimen the exterior negative moment section strains were well below yield at failure.

Even though these plots demonstrate that flexural equilibrium could exist at the failure load levels, they also show that the bending forces may have been high enough to have influenced the shear failures which occurred. This factor must be kept in mind during the remainder of the evaluation of the test results.

The results of any elastic frame analysis should be used very carefully in the evaluation of these structures, however. The portion of the end of the member that is taken as rigid is very important in determining the elastic negative moments. Taking a greater length as rigid increases the negative moment, and taking a shorter length decreases the negative moment. If no rigid portion is assumed, the moments computed at the tips of the fillets will be small, and may be positive at the corners of the structures, depending on the loading pattern considered.

In order to study the influence of the portion of the member assumed to be rigid, additional analyses using Strudl were completed. In these analyses, the length of member assumed to be rigid was either 4.5 in. or zero. Midspan moments and moments at the tips of the fillets, 6.75 in. from the centerline of the supports, are plotted against rigid length in Fig. 8.12. In all cases there is a marked, almost linear, reduction in positive moment with increasing rigid length, and a corresponding increase in negative moments.

For the horizontal members in the 3:1 loading case, Fig. 8.12(a), the exterior negative moments in the end spans are so small that they hardly matter, regardless of the rigid portion assumed. In the same end span, the midspan positive moment exceeds the interior negative moment unless the rigid
length is more than 6 in. In the interior span, the positive moment exceeds the negative moment unless the rigid length is more than 3.5 in.

These values seem to imply that not a great deal of redistribution of moment would be necessary to make the positive and negative moments equal in the interior span, as was assumed when computing the fully plastic moment distributions discussed earlier.

The same values also indicate that a great deal of redistribution would be necessary to make the two negative moments equal to the positive moment in the end span. The required redistribution is so great, for any rigid length considered, that it becomes unlikely. The next most favorable condition is consequently the case of interior negative and midspan positive moments being equal, and with the exterior negative moment somewhat lower. Thus, the minimum moment positions plotted on the interaction diagrams are somewhat optimistic for the end spans but quite reasonable for the interior span.

Figure 8.12(b) indicates that the end vertical members have equal positive and negative moments if the rigid length is 5.2 in. at each end of each member. Regardless of the exact value, if there is any such correct value, of the rigid length, the assumption that midspan positive and negative moments may become equal is not unreasonable, since no large amount of redistribution is required.

The problem of the assumptions to be used in a frame analysis can be avoided by the use of an elastic finite element analysis, but that does not entirely solve the problem. First, the structure is not completely elastic even at working load levels or lower, and secondly, the interpretation of the results to give bending moments and shears is not always simple or straightforward. In the case of the conduit structure, stress concentrations at the
corners of the openings as shown in Figs. 8.3 to 8.5, even with the fillets present, result in non-linear stress distributions across the depths of the negative moment regions which greatly complicate the determination of thrusts and moments at these sections.

The use of the inelastic finite element solution described in Ref. 2 is a partial answer, though the method is more generally suited to the determination of expected behavior of a given structure than to the determination of design moments, shears, and thrusts for which reinforcement must be designed.

The elastic stress distributions shown in Figs. 8.3 and 8.4 do give some vital information about the distributions of moments between positive and negative moments, in spite of the interpretation problems. For the end span, the positive moment is considerably greater than the interior negative moment, and the exterior negative moment is very small, even if somewhat undefined. A number of interpretations of the interior negative moment are possible, but it can be no more than about half the positive moment.

The interior span negative moment is slightly more than half the mid-span positive moment. Again, various interpretations of the negative moment are possible. The peak compressive stresses at the two sections are comparable, even though the moments are clearly somewhat different.

If these moments are examined relative to the results of the conventional frame analyses as plotted in Figs. 8.12(a), it appears that the "correct" rigid lengths in the joint area are perhaps 2.5 in. for the end span and 2 in. for the interior span. Both distances represent only about half the available joint width, indicating that deformations within the joint area have a significant influence on the response of the structure.
With these rigid lengths, the controlling moments are positive rather than negative, but are about the same absolute magnitudes as the maximum moments plotted in the interaction diagrams in Figs. 8.6 to 8.10.

The elastic distribution of stress in the end vertical member which are plotted in Fig. 8.5 indicate that the positive moment is about twice the negative moment at the tip of the fillet, although the stress concentrations at the corner again cloud the interpretation of the exact value of the negative moment.

From Fig. 8.12(b), which shows the results of several conventional frame analyses, a rigid length of about 2 in. results in positive moments approximately twice the negative moment. Thus, an assumption of a rigid length of 2.0 to 2.5 in., or half the distance from the centerline of the column to the face of the column, at each end of each member will bring the results of a frame analysis into reasonable agreement with the results of elastic finite element analyses for this particular structure.

These results are important, when viewed in conjunction with the results of the strain gage readings. The positive moment sections generally had the highest moments and the negative moment sections never reached the yield moments. Consequently, it appears that the failures were basically shear failure with no more than minimal disturbances from flexural stresses or deformations.

8.3 Shear Strength

The prediction of the shear strengths of the test specimens is a much more complex problem than the prediction of the flexural behavior. There is no comprehensive rational theory for shear strength of reinforced concrete
members, and there is little test data from other investigations which is relevant to the current problem, because of the complexity of loading and geometry of the structures.

Nominal shear stresses and shear stresses normalized in terms of $f'_c$ and $\sqrt{f'_c}$ on a number of potential critical sections are listed in Tables 8.1 and 8.2, for specimens failing under vertical and horizontal loads, respectively. The values of $R_3'$ and $R_7'$ shown in Table 8.1 are for the second tests on these specimens, which were conducted after the primary failures of the end vertical members, and thus are for somewhat different loading conditions than the other specimens.

The shear stresses were computed using

$$v_u = \frac{V_u}{bd} \quad (8.2)$$

where

- $v_u$ = nominal shear stress,
- $V_u$ = shear force at failure,
- $b$ = width of member, and
- $d$ = effective depth of tension steel.

The shear force at the face of the support was computed as

$$V_u = \frac{w_u \lambda_n}{2} \quad (8.3)$$

where

- $w_u$ = failure load per unit length of member
- $\lambda_n$ = clear span face to face of columns

Due account was taken of the fact that the specimens were 10 in. wide.
This ignores the fact that the shear force at the interior negative moment section of the end span was slightly greater than this value because of asymmetry of the moment diagram. For the section at the face of the interior column, the shear might be as much as 10 percent higher than the listed values, with the same absolute but a larger relative difference at sections away from the column face. Fig. 8.2 provides some information on this.

Tables 8.1 and 8.2 also list other basic data, including span/depth ratios, concrete strength, loads at failure and at first shear cracking, and the average compression stress due to the axial loads.

As is usually the case with shear failures in concrete, there is a certain amount of scatter in the test data. However, there are a number of things which are quite clear. The horizontal span members, with $\ell_n/d = 4.1$ or less, all developed inclined cracking at loads appreciably below the failure load. The vertical span members, with $\ell_n/d = 5.6$, developed shear cracks and failed without additional load.

The axial compressions acting on the members apparently increased the shear strengths. The margin of increase is not clear from the raw data. Specimens R2 and R3' were identical. R2 resisted 105 k/ft$^2$ while R3' resisted 97.5 k/ft$^2$. The axial compression stress was 790 lb/in.$^2$ in R2 and zero in R3', which seems to indicate only a minor influence of axial load on strength, although the cracking loads were significantly different.

On the other hand, comparisons between R6 and R7', which were quite similar to each other, seems to indicate a large difference in shear as a result of axial compression.
Likewise, depending on the particular pairs of specimens used for the comparison, the steel ratio and the concrete strength can appear to be of either little or great significance.

One thing is evident in all of the values. The shear stresses at failure were all very high. The only current specification applying to deep beams is Sec. 11.9 of the 1971 ACI Code (14). A number of factors are involved in the Code expressions, but the upper limit on shear to be assigned to concrete under design ultimate loads is $6\sqrt{f_c}$. The lowest measured shear stress on the critical section at $0.15 \ell_n$ from the support was $11.7 \sqrt{f_c}$ and the highest was $17.1 \sqrt{f_c}$.

The test values were so much higher than the upper limit on Code values that the Code seems quite irrelevant for these particular structures. Part of the difference may be the presence of the axial loads, but this factor can account for only a portion of the difference, since there was no axial load in some cases.

The Code shear stress depends on the ratio of $M_u/V_u\ell$ and on the steel ratio, $\rho_w$. In the conduit specimens, the steel ratios were generally small, and the upper limit on shear for specimens R4 to R7 is actually about $5.2 \sqrt{f_c}$ rather than $6 \sqrt{f_c}$.

In addition, $\ell_n/d$ is limited to 5, so the vertical members which had $\ell_n/d = 5.6$ do not fit the provisions, and are consequently limited by normal beam shear stresses of about 2 to $2.1 \sqrt{f_c}$, amplified by compression effects to perhaps $3.5 \sqrt{f_c}$. The lowest shear stress at failure was $12.2 \sqrt{f_c}$ for these members.

From an examination of the data from Tables 8.1 and 8.2, there is no consistent pattern with span-depth ratio. The deepest member, R1, was the
The actual shear stress was slightly lower than that in R3' even though R3' had a considerably larger \( \epsilon_n/d \) ratio; neither had appreciable axial compression stresses acting. The values of \( \nu_u/\sqrt{\frac{f_l}{c}} \) are plotted versus \( \epsilon_n/d \) in Fig. 8.13. There is no apparent correlation.

A principal stress analysis of an elastic member subjected to axial compression and shear, and in which it is assumed that failure will occur when the principal tension stress reaches a limiting tension stress, can lead to an expression of the form:

\[
\text{Principal Tension} = \sigma_t = \sqrt{\left(\frac{f_h}{2}\right)^2 + v^2} - \frac{f_h}{2}
\]
This can then be solved to find the shear stress, \( v \), leading to the first tensile crack:

\[
v = f_t \sqrt{1 + \frac{f_h}{f_t}}
\]

This equation then suggests a shear strength equation having the same general form, expressed as:

\[
v_u = v_{uo} \sqrt{1 + \frac{f_h}{f_t}}
\]

where

- \( v_{uo} \) = shear strength when \( f_h = 0 \),
- \( f_h \) = longitudinal stress = \( N_u/A_g \),
- \( f_t \) = tensile strength of the material,
- \( N_u \) = axial thrust, compression positive, and
- \( A_g \) = gross area of member.

The basic shear strength, \( v_{uo} \), can be expressed as \( A \sqrt{f_t} / \sqrt{c} \).

Solving for \( A \), then one obtains

\[
\frac{v_u}{\sqrt{f_t} \sqrt{1 + \frac{f_h}{f_t}}} = A
\]

A is not necessarily a constant, and may be expected to be a function of \( \lambda_n/d \), \( \rho \), the moment/shear ratio, and possibly other variables.
The values of \( \frac{\sqrt{f_{t}}}{\sqrt{f_{c}}} \) are plotted versus \( \varepsilon_{n}/d \) in Fig. 8.15.

In this case, \( f_{t} = 5 \sqrt{f_{c}} \) is taken as a reasonable value. A similar plot was also prepared using \( f_{t} = f_{sp} \), where \( f_{sp} \) is the tensile strength determined from split cylinder tests, as listed in Table 3.2, but there was somewhat more scatter in the data, apparently due to the inconsistencies in the relationship between \( f_{sp} \) and \( f_{t} \), which can be noted in the table.

It was also assumed that \( v_{u0} \) might be a function of \( f'_{c} \) rather than \( \sqrt{f'_{c}} \), and values of \( \frac{v_{u}}{\sqrt{f'_{c}}} \), using \( f_{t} = 5 \sqrt{f'_{c}} \), were plotted versus \( \varepsilon_{n}/d \) in Fig. 8.16. The scatter is not appreciably greater or less than in Fig. 8.15, and either \( f'_{c} \) or \( \sqrt{f'_{c}} \) could be used as the variable relating concrete quality and shear strength, at least within the range of \( f'_{c} \) considered.

In both cases specimen R3' is completely outside the range of the rest of the data, with a shear strength inexplicably high for a member without axial load.

Specimen R1, with \( \varepsilon_{n}/d = 2.5 \), is plotted as a line rather than a point in both cases because it is not known for certain what the horizontal, and hence axial, loads were at failure. If \( f_{h} = 170 \) lb/in.\(^{2} \), the maximum value, the lower value shown is correct. If \( f_{h} = 0 \), the upper value is correct.

Because of the reliance of current design practice on \( \sqrt{f'_{c}} \) as the most reasonable relationship between concrete quality and shear resistance, that variable will be used here.
With reference to Fig. 8.15, it can be seen that a lower bound to the test data occurs if $A = (11.5 - \frac{\lambda_n}{d})$. That is, the minimum shear strength can be expressed as:

$$v_u = (11.5 - \frac{\lambda_n}{d}) \sqrt{\frac{f'_c}{5}} \left(1 + \frac{f_h}{5 \sqrt{f'_c}}\right)$$

(8.4)

where $v_u$ = nominal shear stress on a section $0.15 \lambda_n$ from face of supporting member.

This should be valid for $2 < \frac{\lambda_n}{d} < 6$, $4000 < f'_c < 6000$ lb/in.$^2$, and $f_h < 1300$ lb/in.$^2$.

The range of validity may be larger, but this would have to be confirmed by further testing.

This expression should be used only for structures similar in nature to the test specimens. Continuity would be the most important structural feature. Multiple spans should not be necessary, but rigid corner connections would appear to be. The expression has validity only for uniformly distributed loadings, or other loadings which closely approximate this ideal condition.

The expression should not be applied to simply supported beams nor to members subjected to single or dual concentrated loads as the major loading.

An upper limit to the test data, except for specimen R3', can be expressed as

$$v_{u_{\text{max}}} = (15.75 - 1.625 \frac{\lambda_n}{d}) \sqrt{\frac{f'_c}{5}} \left(1 + \frac{f_h}{5 \sqrt{f'_c}}\right)$$

(8.5)
Equation 8.4 would appear to provide an adequate basis for the shear design of multi-opening conduits subjected to extremely large distributed loadings which produce bending, shear, and axial compression in the main structural members.

There is very little other data on the shear strength of uniformly loaded deep beams to compare with the recommendation made above. The data from two series of tests, Refs. 15 and 16, was analyzed and is plotted in Fig. 8.17, along with Eqs. 8.4 and 8.5. These data points are from tests of simply supported beams without appreciable or known axial loads, and there is no strong reason to expect them to fit particularly well with the results of the conduit tests. As can be seen, there are some points below the Eq. 8.4 values, but the majority are higher. A few comments must be made to provide some perspective.

The shear stresses were evaluated on sections 0.15 \( \Delta_n \) from the centers of the supporting rollers, where \( \Delta_n \) was taken as the initial distance between rollers. The bearing plates were of significant width, but the face-to-face distance between bearing plates does not appear to be a reasonable span value for a simply supported member.

There may have been significant axial compressions present in both series of tests. De Paiva's beams (15) were supported on 2 in. diam rollers 4.5 in. in length, and were subjected to a maximum reaction of 140 kips. Crist's beams (16) were supported on 4 in. diam rollers 8 to 12 in. long, and had a maximum reaction of about 375 kips, on an 8 in. length. Even a small rolling coefficient of friction must result in an appreciable, if unknown, longitudinal compression. These forces have been ignored, but they must have been present in these and many other test specimens. In the current test
series, the axial forces were known for the horizontal members and can be closely estimated for the vertical members.

Points from de Paiva's tests (15) are plotted as squares. Open squares represent flexural failures while closed squares represent all others, including diagonal tension, shear-compression, web crushing, and splitting. All of these points falling below the Eq. 8.4 value are for flexural failures. All shear related failures lie above the line, as indeed do several of the flexural failures.

Points from Crist's tests are plotted as triangles, again with open triangles representing flexural failures and solid triangles all others. A number of points lie well below the Eq. 8.4 line, but there are definite reasons for most of them. The three low values plotted at \( \frac{a_n}{d} = 1.7 \) are for specimens which suffered bearing stress failures, apparently at least partially due to inadequate size of bearing plates. The low value at \( \frac{a_n}{d} = 3.6 \) was for a shear-compression failure with no detail problems.

Several of Crist's specimens had shear reinforcement, in the form of vertical or horizontal stirrups. The effectiveness of this steel has not been fully established by these tests. Considering the group of beams with \( \frac{a_n}{d} = 2.7 \), the highest absolute and normalized shear stress was for a beam with no web reinforcement: It is the scatter in the flexural strengths which becomes difficult to explain in this case. Shear reinforcement was undoubtedly effective in the \( \frac{a_n}{d} = 3.6 \) specimens but its usefulness must be questioned for the deeper members.

Crist's specimens all carried loads much higher than that causing first shear cracking, usually at least 2.5 times the cracking load. This aspect of the behavior was much different than in the conduit specimens.
In view of the considerable differences between the test specimens, the agreement between Eq. 8.4 and the results of tests described in Refs. 15 and 16 is unexpectedly good. Most of the data points actually lie between the bounds established by Eq. 8.4 and 8.5.

The relatively good agreement between the results of the various tests supports the validity of using Eq. 8.4 as a basis for the shear design of thick-walled conduits. Either Eq. 8.4 itself or an equation representing a parallel line at a slightly lower stress level is recommended for design
9. SUMMARY AND CONCLUSIONS

This report is the second concerned with physical tests on relatively large scale models of multiple opening reinforced concrete conduits which have been designed for earth pressure loads as large as 30 k/ft\(^2\). A prototype is shown in Fig. 1.1 and the models in Figs. 2.1 and 2.2.

The results of the tests of the final five specimens of a series of eight are presented in detail. Detailed descriptions of the first three tests are contained in Ref. 1. The results of all tests are summarized in Table 1.1.

After the initial test on a thicker-walled specimen, the tests were used to explore the influence of loading pattern, concrete strength, and reinforcement quantity on the strength and behavior of the specimens. Loads as high as 150 k/ft\(^2\) in the vertical direction were reached, in the first specimen, R1. Maximum loads of 50 to 132.5 k/ft\(^2\) were reached in later specimens, depending on the loading pattern being applied.

The distributions of moments and shears in the members can be found to satisfactory accuracy by conventional frame analysis methods, provided that stiffnesses of the ends of the members are defined as suggested in Chapter 8. A two-dimensional elastic finite element analysis could also be used to obtain design forces, though there are some interpretation problems with this approach.

The flexural strengths of the members, which are also subjected to large axial compression forces, can be computed using normal moment-thrust interaction curve techniques, as described in Chapter 8.

The shear strengths governed in all eight test specimens. The shear strengths were all quite high, because of the low span-depth ratios, and

-57-
were further enhanced by the presence of the axial compression forces acting on the members. A lower bound to the nominal shear stress at failure is proposed, Eq. 8.4:

$$v_u = (11.5 - \frac{n}{d}) \sqrt{f_c} \sqrt{1 + \frac{f_h}{5 \sqrt{f_c}}}$$

This equation also forms a reasonable lower bound to the results of the few other tests of deep beams subjected to distributed loadings which have been reported in the literature.

The results of these tests provide a sound basis for the design of multiple opening reinforced concrete conduits for nominal earth pressure loads up to 30 to 35 k/ft², or for fill heights of up to about 300 ft.
LIST OF REFERENCES


14. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-71)," American Concrete Institute, Detroit, 1971.


# TABLE I.1 SUMMARY OF TEST RESULTS

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>MEMBER DEPTH, in.</th>
<th>CONCRETE STRENGTH, psi</th>
<th>BAR SIZE</th>
<th>TENSION STEEL, PERCENTAGE, %</th>
<th>NOMINAL LOAD, RATIO VERT./HORIZ.</th>
<th>FAILURE LOAD, ksf VERT./HORIZ.</th>
<th>FAILURE PATTERN</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1</td>
<td>13.5 EXTERN, 9.0 INTERN.</td>
<td>6250</td>
<td>7</td>
<td>1.0 EXTERNAL 1.7 INTERNAL</td>
<td>3:1</td>
<td>150 &lt; 10**</td>
<td>SHEAR</td>
</tr>
<tr>
<td>R2</td>
<td>9.0</td>
<td>5520</td>
<td>6</td>
<td>1.2</td>
<td>3:1</td>
<td>105 35</td>
<td>SHEAR</td>
</tr>
<tr>
<td>R3</td>
<td>9.0</td>
<td>5520</td>
<td>6</td>
<td>1.2</td>
<td>1:1</td>
<td>67.5 67.5</td>
<td>SHEAR</td>
</tr>
<tr>
<td>R4</td>
<td>9.0</td>
<td>5800</td>
<td>4</td>
<td>0.55</td>
<td>3:1</td>
<td>1325 40</td>
<td>SHEAR-FLEXURE</td>
</tr>
<tr>
<td>R5</td>
<td>9.0</td>
<td>5300</td>
<td>4</td>
<td>0.55</td>
<td>1:1</td>
<td>75 75</td>
<td>SHEAR</td>
</tr>
<tr>
<td>R6</td>
<td>9.0</td>
<td>4550</td>
<td>4</td>
<td>0.55</td>
<td>3:1</td>
<td>105 35</td>
<td>SHEAR</td>
</tr>
<tr>
<td>R7</td>
<td>9.0</td>
<td>4450</td>
<td>4</td>
<td>0.55</td>
<td>1:1</td>
<td>60 60</td>
<td>SHEAR</td>
</tr>
<tr>
<td>R8</td>
<td>9.0</td>
<td>5025</td>
<td>3</td>
<td>0.3</td>
<td>3:1</td>
<td>105 35</td>
<td>SHEAR-FLEXURE</td>
</tr>
</tbody>
</table>

* PROPERTIES DIFFERED FOR EXTERNAL AND INTERNAL MEMBERS IN R1 ONLY
** HORIZONTAL LOAD WAS REDUCED TO OBTAIN FAILURE
## TABLE 2.1
Nominal Member Thicknesses and Tension Steel Ratios

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Specimen Thickness-in.</th>
<th>R1</th>
<th>R2 and R3</th>
<th>R4 through R7</th>
<th>R8</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Total Depth-in.</strong></td>
<td>13.5</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
<td>9.0</td>
</tr>
<tr>
<td><strong>External Effective Members</strong></td>
<td>11.56</td>
<td>7.13</td>
<td>7.25</td>
<td>7.31</td>
<td></td>
</tr>
<tr>
<td><strong>Internal Effective Members</strong></td>
<td>7.06</td>
<td>7.13</td>
<td>7.25</td>
<td>7.31</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Specimen Thickness-in.</th>
<th>Tension* Steel ratio, %</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Total Depth-in.</strong></td>
<td>9.0</td>
<td>1.0</td>
</tr>
<tr>
<td><strong>External Effective Members</strong></td>
<td>7.13</td>
<td>1.2</td>
</tr>
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<td><strong>Internal Effective Members</strong></td>
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<td>0.55</td>
</tr>
<tr>
<td><strong>Specimen Thickness-in.</strong></td>
<td>10</td>
<td>10</td>
</tr>
</tbody>
</table>

*Tension Steel Area is Equal to Compression Steel Area*
### TABLE 3.1
Reinforcement Strength Properties

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>$f_y$ - k/in.$^2$</th>
<th>$f_{ult}$ - k/in.$^2$</th>
<th>$e_u$ - % in 8 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3</td>
<td>78.0</td>
<td>124.0</td>
<td>12.2 %</td>
</tr>
<tr>
<td>#4</td>
<td>70.0</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>#6</td>
<td>72.7</td>
<td>118.9</td>
<td>14.7 %</td>
</tr>
<tr>
<td>#7</td>
<td>69.8</td>
<td>112.2</td>
<td>13.6 %</td>
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</tbody>
</table>

Each value average of three tests

### TABLE 3.2
Concrete Strength Properties

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Slump in.</th>
<th>Age-days (cylinders)</th>
<th>$f'_c$ - lb/in.$^2$</th>
<th>$f_{sp}$ - lb/in.$^2$</th>
<th>$E_c$ - lb/in.$^2$ (initial Modulus)</th>
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</thead>
<tbody>
<tr>
<td>R1</td>
<td>2 1/4</td>
<td>129</td>
<td>6,250</td>
<td>360</td>
<td>$4.10 \times 10^6$</td>
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<tr>
<td>R2</td>
<td>2 3/4</td>
<td>66</td>
<td>5,520</td>
<td>338</td>
<td>$3.92 \times 10^6$</td>
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<td>R3</td>
<td>2</td>
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<td>5,520</td>
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<td>$3.95 \times 10^6$</td>
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<tr>
<td>R4</td>
<td>-</td>
<td>86</td>
<td>5,800</td>
<td>470</td>
<td>$3.84 \times 10^6$</td>
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<tr>
<td>R5</td>
<td>-</td>
<td>34</td>
<td>5,300</td>
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<td>4,550</td>
<td>461</td>
<td>$3.73 \times 10^6$</td>
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<tr>
<td>R7</td>
<td>2 3/4</td>
<td>28</td>
<td>4,450</td>
<td>348</td>
<td>$3.65 \times 10^6$</td>
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<tr>
<td>R8</td>
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<td>38</td>
<td>5,025</td>
<td>407</td>
<td>$4.40 \times 10^6$</td>
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<td>Load No.</td>
<td>Vertical, k/ft²</td>
<td>Horizontal Load</td>
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<td>----------------</td>
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<tr>
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<td>Ext. Axial</td>
<td>Int. Axial</td>
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<td>132.5</td>
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<td>19</td>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

* Only vertical span loads were increased after Load No. 15. Horizontal jacks were locked, and loads increased due to deflections of test specimen.
<table>
<thead>
<tr>
<th>Load No.</th>
<th>Vertical, k/ft²</th>
<th>Horizontal Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ext. Axial</td>
<td>Int. Axial</td>
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<td>2</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>3</td>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>4</td>
<td>37.5</td>
<td>37.5</td>
</tr>
<tr>
<td>5</td>
<td>45.0</td>
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<tr>
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TABLE 7.3
Test R6, Nominal Equivalent Pressures, k/ft²

<table>
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<tr>
<th>Load No.</th>
<th>Vertical, k/ft²</th>
<th>Horizontal Load</th>
<th></th>
</tr>
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<td>Ext. Axial</td>
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<tr>
<td>14</td>
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</tbody>
</table>
TABLE 7.4
Test R7, Nominal Equivalent Pressures, k/ft$^2$

<table>
<thead>
<tr>
<th>Load No.</th>
<th>Vertical, k/ft$^2$</th>
<th>Horizontal Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ext. Axial</td>
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<td>82.5</td>
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<tr>
<td>12</td>
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</tbody>
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TABLE 7.5
Test R8, Nominal Equivalent Pressures, k/ft²

<table>
<thead>
<tr>
<th>Load No.</th>
<th>Vertical, k/ft²</th>
<th>Horizontal Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ext. Axial</td>
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<td>97.5</td>
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<tr>
<td>13</td>
<td>105.0</td>
<td>105.0</td>
</tr>
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<td>&lt;105.0</td>
</tr>
<tr>
<td>15</td>
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<td>0</td>
</tr>
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Table 8.1 Shear in Specimens Failing under Vertical Loads

<table>
<thead>
<tr>
<th>Spec.</th>
<th>$k_n/d$</th>
<th>$f_c'$</th>
<th>$w_{vu}$</th>
<th>$w_{vc}$</th>
<th>$w_{hu}$</th>
<th>$N_u/A_g$</th>
<th>$v_u$</th>
<th>$v_T$</th>
<th>$v_f$</th>
<th>$v_{fu}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1</td>
<td>2.53</td>
<td>6250</td>
<td>0.01</td>
<td>150</td>
<td>112.5</td>
<td>&lt;10</td>
<td>1320</td>
<td>0.211</td>
<td>16.7</td>
<td>920</td>
</tr>
<tr>
<td>R2</td>
<td>4.11</td>
<td>5520</td>
<td>0.012</td>
<td>105</td>
<td>82.5</td>
<td>35</td>
<td>1495</td>
<td>0.271</td>
<td>20.1</td>
<td>1045</td>
</tr>
<tr>
<td>R3'</td>
<td>4.11</td>
<td>5520</td>
<td>0.012</td>
<td>97.5</td>
<td>&lt;67.5</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1390</td>
<td>0.252</td>
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<tr>
<td>R4</td>
<td>4.03</td>
<td>5800</td>
<td>0.0055</td>
<td>132.5</td>
<td>82.5</td>
<td>40</td>
<td>1855</td>
<td>0.320</td>
<td>21.4</td>
<td>1300</td>
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<td>R6</td>
<td>4.03</td>
<td>4550</td>
<td>0.0055</td>
<td>105</td>
<td>67.5</td>
<td>35</td>
<td>1470</td>
<td>0.323</td>
<td>21.8</td>
<td>1030</td>
</tr>
<tr>
<td>R7'</td>
<td>4.03</td>
<td>4450</td>
<td>0.0055</td>
<td>82.5</td>
<td>67.5</td>
<td>24.6*</td>
<td>555</td>
<td>1155</td>
<td>0.260</td>
<td>17.3</td>
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<tr>
<td>R8</td>
<td>4.00</td>
<td>5025</td>
<td>0.003</td>
<td>105</td>
<td>82.5</td>
<td>35</td>
<td>1460</td>
<td>0.290</td>
<td>20.6</td>
<td>1020</td>
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</tbody>
</table>

* Equivalent distributed load, loads actually on horizontal members only.

\[
\begin{align*}
  w_{vu} &= \text{Vertical load at failure} \\
  w_{vc} &= \text{Vertical load at shear cracking} \\
  w_{hu} &= \text{Horizontal load at failure}
\end{align*}
\]
<table>
<thead>
<tr>
<th>Spec.</th>
<th>$l_n/d$</th>
<th>$f'_c$ [lb/in.²]</th>
<th>$w_{vu}$ [k/ft²]</th>
<th>$w_{hu}$ [k/ft²]</th>
<th>$w_{hc}$ [k/ft²]</th>
<th>$N_u/A_g$ [lb/in.²]</th>
<th>$v_u$ [lb/in.²]</th>
<th>$v'_u$ [lb/in.²]</th>
<th>$f'_C$ [lb/in.²]</th>
<th>$f''_C$ [lb/in.²]</th>
<th>$v_u$ [lb/in.²]</th>
<th>$v'_u$ [lb/in.²]</th>
<th>$f'_C$ [lb/in.²]</th>
<th>$f''_C$ [lb/in.²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>R3</td>
<td>5.68</td>
<td>5520</td>
<td>0.012</td>
<td>67.5</td>
<td>67.5</td>
<td>60</td>
<td>1230</td>
<td>0.241</td>
<td>17.9</td>
<td>935</td>
<td>0.169</td>
<td>12.6</td>
<td>1185</td>
<td>0.215</td>
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<td>R5</td>
<td>5.59</td>
<td>5300</td>
<td>0.0055</td>
<td>75</td>
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<td>0.244</td>
</tr>
<tr>
<td>R7</td>
<td>5.59</td>
<td>4450</td>
<td>0.0055</td>
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<td>60</td>
<td>60</td>
<td>1090</td>
<td>0.261</td>
<td>17.4</td>
<td>815</td>
<td>0.183</td>
<td>12.2</td>
<td>1035</td>
<td>0.232</td>
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</tbody>
</table>

$w_{vu}$ = Vertical load at failure
$w_{hu}$ = Horizontal load at failure
$w_{hc}$ = Horizontal load at shear cracking

Table 8.2 Shear in Specimens Failing under Horizontal Loads

Diagram showing load distribution and failure points at support and tip of fillet.
Note:
Construction Joints and Piping Locations not shown.

FIG. 1.1 CROSS-SECTION OF CONDUIT THROUGH DAM AT FALL CREEK RESERVOIR, OREGON
FIG. 2.1 DIMENSIONS OF CONDUIT SPECIMEN R1
FIG. 2.2 DIMENSIONS OF SPECIMENS R2 THROUGH R8
FIG. 2.3 ARRANGEMENT OF REINFORCEMENT IN SPECIMEN R1

Denotes: 7 - #3 bars, Type 01 High Strength (Grade 60)
FIG. 2.4 DETAILS OF REINFORCEMENT, SPECIMEN R1
FIG. 2.5 ARRANGEMENT OF REINFORCEMENT IN SPECIMENS R2 THROUGH R8

Note: H4 used in place of H6 in R4 through R7
H3 used in place of H6 in R8

Note: Clear Cover: 1\(\frac{1}{2}\)"
To All Side Faces
FIG. 2.6 DETAILS OF REINFORCEMENT, SPECIMENS R2 THROUGH R8
FIG. 2.7 PHOTOGRAPH OF CORNER REINFORCEMENT DETAIL, SPECIMEN R1

FIG. 2.8 PHOTOGRAPH OF REINFORCEMENT AT INTERIOR COLUMN-EDGE MEMBER JOINT, SPECIMEN R1
FIG. 2.9 PHOTOGRAPH OF SPECIMEN R1 REINFORCEMENT IN FORMWORK
FIG. 3.1 TYPICAL STRESS-STRAIN RELATIONSHIPS FOR REINFORCING STEEL
FIG. 3.2 REPRESENTATIVE STRESS-STRAIN CURVES FOR CONCRETE FROM TEST SPECIMENS
FIG. 4.1 ARRANGEMENT OF LOADING UNITS
FIG. 4.2 PHOTOGRAPH OF SPECIMEN R1 IN TEST FRAME BEFORE ASSEMBLY OF LEADING EQUIPMENT
FIG. 4.3 PHOTOGRAPH OF SPECIMEN R1 WITH LOADING EQUIPMENT IN PLACE
FIG. 4.4 SPACING OF LOADS FOR SPECIMEN R1

Note: All Jacks Are In Pairs
FIG. 4.5 LOCATIONS OF LOADING UNITS FOR SPECIMENS R2 to R8
Strain gages attached to top layer of steel unless noted otherwise.

Gages on Bottom Steel

FIG. 5.1 LOCATIONS OF STRAIN GAGES IN SPECIMEN R1
Strain gages attached to upper layer of steel unless otherwise noted.

FIG. 5.2 LOCATIONS OF STRAIN GAGES IN SPECIMEN R2, R4, R6 and R8
Strain gages attached to upper layer of steel unless otherwise noted.

FIG. 5.3 LOCATIONS OF STRAIN GAGES IN SPECIMEN R3, R5, AND R7
FIG. 5.4  LOCATIONS OF STRAIN GAGES ON CONCRETE SURFACES OF SPECIMENS R5 AND R7
Fig. 5.5 Locations of strain gages on concrete surfaces of specimens R6 and R8
FIG. 5.6 LOCATIONS AND DESIGNATIONS OF DEFLECTION GAGES
FIG. 7.1 PHOTOGRAPH OF SPECIMEN R4 AFTER TEST TO FAILURE

FIG. 7.2 FAILED END SPAN OF SPECIMEN R4
Fig. 7.3 LOAD-STRAIN CURVES, EXT. NEGATIVE MOMENT SECTION, R4
FIG. 7.4 LOAD-STRAIN CURVES, END SPAN POSITIVE MOMENT SECTION, R4
Reinforcement Strain

FIG. 7.5 LOAD-STRAIN CURVES, INT. NEGATIVE MOMENT SECTION, R4
FIG. 7.7 LOAD-STRAIN CURVES, INTERIOR SPAN POSITIVE MOMENT, R4
FIG. 7.8 LOAD-STRAIN CURVES, EXTERIOR VERTICAL MEMBER, NEGATIVE MOMENTS, R4
Reinforcement Strain

FIG. 7.9 LOAD-STRAIN CURVES, EXTERIOR VERTICAL MEMBER, POSITIVE MOMENTS, R4
FIG. 7.10 LOAD-STRAIN CURVES, END OF INTERIOR VERTICAL MEMBER, R4
FIG. 7.11 LOAD-STRAIN CURVES, MIDHEIGHT OF INTERIOR VERTICAL MEMBER, R4
FIG. 7.12  LOAD-HORIZONTAL DEFLECTION CURVE, END OPENING IN FAILURE ZONE, R4
FIG. 7.13 LOAD-HORIZONTAL DEFLECTION CURVE, CENTRAL OPENING, R4
Horizontal Deflection
FIG. 7.14 LOAD-HORIZONTAL DEFORMATION CURVE, END OPENING, R4
FIG. 7.15 LOAD-VERTICAL DEFLECTION CURVE, END OPENING IN FAILURE ZONE, R4
Vertical Deflection

FIG. 7.16 LOAD-VERTICAL DEFLECTION CURVE, CENTRAL OPENING, R4
FIG. 7.17 LOAD-VERTICAL DEFLECTION, END OPENING, R4
FIG. 7.18 PHOTOGRAPH OF SPECIMEN R5 AFTER TEST TO FAILURE

FIG. 7.19 FAILED EXTERIOR VERTICAL MEMBER OF SPECIMEN R5
FIG. 7.20 LOAD-STRAIN CURVES, EXTERIOR NEGATIVE MOMENT SECTION, R5
FIG. 7.21 LOAD-STRAIN CURVES, END SPAN POSITIVE MOMENT SECTION, R5
FIG. 7.22 LOAD-STRAIN CURVES, INTERIOR NEGATIVE MOMENT SECTION, R5
Vertical Applied Load, kips/ft²

FIG. 7.23 LOAD-STRAIN CURVES, INTERIOR SPAN NEGATIVE MOMENT, R5

Compression

Reinforcement Strain, in/in

Tension

0.00040

Gage 11

Gage 12

0 20 40 60 80 100 120 140

0 20 40 60 80 100 120 140
FIG. 7.24 LOAD-STRAIN CURVES, INTERIOR SPAN POSITIVE MOMENT SECTION, R5
FIG. 7.25 LOAD-STRAIN CURVES, EXTERIOR VERTICAL MEMBER, NEGATIVE MOMENTS, R5
FIG. 7.26   LOAD-STRAIN CURVES, EXTERIOR VERTICAL MEMBER, POSITIVE MOMENT SECTION, R5
FIG. 7.27  LOAD-STRAIN CURVES, END OF INTERIOR VERTICAL MEMBER, R5
FIG. 7.28 LOAD-STRAIN CURVES, MIDHEIGHT OF INTERIOR VERTICAL MEMBER, R5
FIG. 7.29 LOAD-HORIZONTAL DEFLECTION CURVE, END OF OPENING, R5
FIG. 7.30 LOAD-HORIZONTAL DEFLECTION CURVE, CENTRAL OPENING, R5
FIG. 7.31 LOAD-HORIZONTAL DEFLECTION CURVE, END OPENING IN FAILURE ZONE, R5
FIG. 7.32 LOAD-VERTICAL DEFLECTION CURVE, END OPENING, R5
FIG. 7.33 LOAD-VERTICAL DEFLECTION CURVE, CENTRAL OPENING, R5
Vertical Deflection, in.

7.34 LOAD-VERTICAL DEFLECTION CURVE, END OPENING IN FAILURE ZONE, R5
FIG. 7.35 PHOTOGRAPH OF SPECIMEN R6 AFTER TEST TO FAILURE

FIG. 7.36 FAILED END SPAN OF SPECIMEN R6
FIG. 7.37 LOAD-STRAIN CURVES, EXTERIOR NEGATIVE MOMENT SECTION, R6
FIG. 7.38 LOAD-STRAIN CURVES, END SPAN POSITIVE MOMENT SECTION, R6
FIG. 7.40 LOAD-STRAIN CURVES, INTERIOR SPAN NEGATIVE MOMENT, R6
FIG. 7.41 LOAD-STRAIN CURVES, INTERIOR SPAN POSITIVE MOMENT SECTION, R6
FIG. 7.42 LOAD-STRAIN CURVES, EXTERIOR VERTICAL MEMBER, NEGATIVE MOMENTS, R6
FIG. 7.43 LOAD-STRAIN CURVES, EXTERIOR VERTICAL MEMBER, POSITIVE MOMENTS, R6
FIG. 7.44 LOAD-STRAIN CURVES, END OF INTERIOR VERTICAL MEMBER, R6
FIG. 7.45 LOAD-STRAIN CURVES, MIDHEIGHT OF INTERIOR VERTICAL MEMBER, R6
FIG. 7.46 LOAD-HORIZONTAL DEFLECTION CURVE, END OPENING IN FAILURE ZONE, R6
FIG. 7.47   LOAD-HORIZONTAL DEFLECTION CURVE, CENTRAL OPENING, R6
FIG. 7.48 LOAD-HORIZONTAL DEFLECTION CURVE, END OPENING, R6
FIG. 7.49 LOAD-VERTICAL DEFLECTION CURVE, END OPENING IN FAILURE ZONE, R6
FIG. 7.50 LOAD-VERTICAL DEFLECTION CURVE, CENTRAL OPENING, R6
FIG. 7.51 LOAD-VERTICAL DEFLECTION CURVE, END OPENING, R6
FIG. 7.52 PHOTOGRAPH OF SPECIMEN R7 AFTER TEST TO FAILURE

FIG. 7.53 FAILED VERTICAL END MEMBER IN SPECIMEN R7
FIG. 7.54 FAILED INTERIOR SPAN IN SPECIMEN R7
AFTER SECOND TEST TO FAILURE
At Reduction Of Horizontal Loads

FIG. 7.55 LOAD-STRAIN CURVES, EXTERIOR NEGATIVE MOMENT SECTION, R7
FIG. 7.56 LOAD-STRAIN CURVES, END SPAN POSITIVE MOMENT SECTION, R7

Reinforcement Strain, in/in

Vertical Applied Load, kips/ft²

Compression  |  Tension

0.00040
FIG. 7.57 LOAD-STRAIN CURVES, INTERIOR NEGATIVE MOMENT SECTION, R7
FIG. 7.58 LOAD-STRAIN CURVE, INTERIOR SPAN NEGATIVE MOMENT SECTION, R7
FIG. 7.59  LOAD-STRAIN CURVES, INTERIOR SPAN POSITIVE MOMENT SECTION, R7
FIG. 7.60 LOAD- STRAIN CURVE, EXTERIOR VERTICAL MEMBER, NEGATIVE MOMENT, R7
FIG. 7.61  LOAD-STRAIN CURVES, EXTERIOR VERTICAL MEMBER, POSITIVE MOMENTS, R7
FIG. 7.62. LOAD-STRAIN CURVES, END OF INTERIOR VERTICAL MEMBER, R7
FIG. 7.63 LOAD-STRAIN CURVES, MIDHEIGHT OF INTERIOR VERTICAL MEMBER, R7
FIG. 7.64 LOAD-HORIZONTAL DEFLECTION CURVE, END OPENING, R7
FIG. 7.65 LOAD-HORIZONTAL DEFLECTION CURVE, CENTRAL OPENING, R7
FIG. 7.66 LOAD-HORIZONTAL DEFLECTION CURVE, END OPENING IN FAILURE ZONE, R7
FIG. 7.67 LOAD-VERTICAL DEFLECTION CURVE, END OPENING, R7
FIG. 7.68 LOAD-VERTICAL DEFLECTION CURVE, CENTRAL OPENING, R7
FIG. 7.69 LOAD-VERTICAL DEFLECTION CURVE, END OPENING IN FAILURE ZONE, R7
FIG. 7.70 PHOTOGRAPH OF SPECIMEN R8 AFTER TEST TO FAILURE

FIG. 7.71 PRIMARY END SPAN FAILURE IN SPECIMEN R8
FIG. 7.72  SECONDARY END SPAN FAILURE IN SPECIMEN R8
FIG. 7.73 LOAD-STRAIN CURVES, EXTERIOR NEGATIVE MOMENT SECTION, R8
FIG. 7.74 LOAD-STRAIN CURVES, END SPAN POSITIVE MOMENT SECTION, R8
FIG. 7.75  LOAD-STRAIN CURVES, INTERIOR NEGATIVE MOMENT SECTION, R8
FIG. 7.76 LOAD-STRAIN CURVES, INTERIOR SPAN NEGATIVE MOMENT, R8
FIG. 7.77 LOAD-STRAIN CURVES, INTERIOR SPAN POSITIVE MOMENT SECTION, R8
FIG. 7.78: LOAD-STRAIN CURVES, EXTERIOR VERTICAL MEMBER, NEGATIVE MOMENTS, R8
FIG. 7.79 LOAD-STRAIN CURVES, EXTERIOR VERTICAL MEMBER, POSITIVE MOMENTS, R8
FIG. 7.80 LOAD-STRAIN CURVE, END OF INTERIOR VERTICAL MEMBER, R8
FIG. 7.81 LOAD-STRAIN CURVE, MIDHEIGHT OF INTERIOR VERTICAL MEMBER, R8
FIG. 7.82 LOAD-HORIZONTAL DEFLECTION CURVE, END OPENING, R8
Fig. 7.83  Load-Horizontal Deflection Curve, Central Opening, R8
FIG. 7.84 LOAD-HORIZONTAL DEFLECTION CURVE, END OPENING IN FAILURE ZONE, R8
FIG. 7.85 LOAD-VERTICAL DEFLECTION CURVE, END OPENING, R8

Vertical Deflection, in.
FIG. 7.86  LOAD-VERTICAL DEFLECTION CURVE, CENTRAL OPENING, R8
FIG. 7.87 LOAD-VERTICAL DEFLECTION CURVE, END OPENING IN FAILURE ZONE, R8.
FIG. 8.1 DISTRIBUTION OF VERTICAL STRESSES ON VARIOUS HORIZONTAL PLANES, EQUIVALENT CONCENTRATED LOADS, R4

$W_v = 3 \text{k/ft}^2$

$W_h = 1 \text{k/ft}^2$
Uniformly Distributed Load

Equivalent Concentrated Loads

$w_v = 3 \text{ k/ft}^2$

$w_h = 1 \text{ k/ft}^2$

FIG. 8.2  DISTRIBUTIONS OF SHEAR STRESS ON HORIZONTAL PLANE IN END SPAN, R4
FIG. 8.3 DISTRIBUTION OF HORIZONTAL STRESSES ON VARIOUS VERTICAL PLANES IN END SPAN, R4
At Midspan

Tension

Tension

At Tip Of Fillet

$W_v = 3k/ft^2$

$W_h = 1k/ft^2$

FIG. 8.4 DISTRIBUTION OF HORIZONTAL STRESSES ON VERTICAL SECTIONS IN INTERIOR SPAN, R4
FIG. 8.5 DISTRIBUTION OF VERTICAL STRESSES ON HORIZONTAL SECTIONS IN END VERTICAL MEMBER, R5
FIG. 8.6 MOMENT-THRUST INTERACTION CURVE FOR SPECIMEN R1
FIG. 8.8 MOMENT-THRUST INTERACTION CURVES FOR SPECIMENS R4 AND R5
FIG. 8.9 MOMENT-THRUST INTERACTION CURVES FOR SPECIMENS R6 AND R7
FIG. 8.10  MOMENT-THRUST INTERACTION CURVE FOR SPECIMEN R8
Comp. Face

Compressional Face

Centroidal Axis

Tension Face

\[ \epsilon_{cu} = 0.003 \]

\[ \epsilon_{su} = 0.003 \]

\[ \epsilon'_{su} = 0.003 \frac{(d - k_u d)'}{k_u d} \]

\[ C_u = A_s' \epsilon_{su} E_s \leq A_s' f_y \]

\[ C_c = 0.85 f'_c \alpha b \]

\[ T = A_s \epsilon_{su} E_s \leq A_s f_y \]

Section \( (A_s' = A_s) \)

Strains

Stresses and Forces

Thrust = \( C_i + C_c - T \)

\[ \Sigma M_{\text{centroid}} = C_s \left( \frac{h}{2} - d' \right) + C_c \left( \frac{h}{2} - \frac{a}{2} \right) + T \left( d - \frac{h}{2} \right) \]

\[ \beta_i = 0.85 \text{ if } f'_c \leq 4,000 \text{ lb/in.}^2 \]

\[ \beta_i = 0.85 - 0.05 \left( \frac{f'_c - 4,000}{1,000} \right) \text{ if } f'_c > 4,000 \text{ lb/in.}^2 \]

FIG. 8.11 SUMMARY OF METHOD OF CALCULATION OF MOMENT-THRUST INTERACTION DIAGRAMS
FIG. 8.12 EFFECTS OF RIGID LENGTHS AT ENDS OF MEMBERS ON BENDING MOMENT DISTRIBUTIONS, CONVENTIONAL FRAME ANALYSIS
FIG. 8.13 BASIC SHEAR STRENGTH DATA PLOTTED VERSUS SPAN-DEPTH RATIO
(a) Freebody of Small Element With Shear and Compressive Stresses

(b) Mohr's Circle Representation of Stresses on Element

Fig. 8.14 Principal Stress Analysis Using Mohr's Circle
FIG. 8.15 NORMALIZED SHEAR STRENGTH DATA, INCLUDING THRUST EFFECTS, VERSUS SPAN-DEPTH RATIO
FIG. 8.16 SHEAR STRENGTH DATA NORMALIZED IN TERMS OF $f'_c$ VERSUS SPAN-DEPTH RATIO
FIG. 8.17 NORMALIZED SHEAR STRENGTH FROM OTHER TESTS VERSUS SPAN-DEPTH RATIO

\[
\frac{v_u}{\sqrt{f_C}} = (15.75 - 1.625 \frac{l_n}{d}) \sqrt{f_C} \sqrt{1 + \frac{f_h}{5\sqrt{f_C}}} 
\]

(Eq. 8.5)

\[
\frac{v_u}{\sqrt{f_C}} = (11.5 - \frac{l_n}{d}) \sqrt{f_C} \sqrt{1 + \frac{f_h}{5\sqrt{f_C}}} 
\]

(Eq. 8.4)
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