STRENGTH AND BEHAVIOR OF SIMULATED CULVERT SECTIONS FAILING IN SHEAR

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I. INTRODUCTION

1. Object and Scope

The object of this investigation was to explore the behavior and strength in shear of reinforced concrete frames, simulating culvert sections, under various combinations of uniform and axial loads. Eleven frames were tested, eight with no web reinforcement in the main member, and three with web reinforcement.

The major variables studied were: ratio of positive to negative moment, ratio of moment to axial load, ratio of vertical to horizontal load, steel percentage, span length, continuity of negative steel, and unintentional differences in concrete strength.

The ultimate application of the results of this investigation is to the design of reinforced concrete box culverts. These tests are those designated as Series B.1 in "A Suggested Program of Tests for the Development of Criteria for the Structural Design of Reinforced Concrete Box Culverts" (1)*

2. Acknowledgments

This project was carried out in the Structural Research Laboratory of the Department of Civil Engineering at the University of Illinois in cooperation with the Ohio River Division Laboratories, Corps of Engineers, U. S. Army, under contract DA-33-017-CIVENG-57-2.

The project was under the direction of Dr. C. P. Siess, Research Professor of Civil Engineering.

* Numbers in parentheses refer to corresponding entries in the list of references.
Appreciation is expressed to G. W. Armstrong, Research Assistant in Civil Engineering and L. W. Hughart, Junior in Civil Engineering, for their aid in conducting the tests and in preparing this report. Mr. Wyck McKenzie, Junior Laboratory Mechanic, was very helpful in fabricating test specimens and equipment.

3. Notation

The following notation has been used in this report:

**Distances**

- \( b \) = width of frame
- \( d \) = effective depth of reinforcement
- \( h \) = distance between mid-depth of horizontal member and point of application of axial load
- \( k_d \) = depth of compression zone at flexural failure
- \( L \) = overall span length
- \( l \) = clear span
- \( x \) = horizontal distance from the critical section to nearest end of frame

**Forces**

- \( N \) = axial load
- \( N_c \) = axial load at diagonal tension cracking
- \( V \) = shearing force
- \( V_c \) = shearing force at critical section, at diagonal tension cracking
- \( W \) = total vertical load
- \( W_c \) = total vertical load at diagonal tension cracking
Moments

\[ M_{1/2} = \text{midspan moment at diagonal tension cracking} \]

\[ M_{st} = \text{total static moment between column face sections} \]

Stresses

\[ f' = \text{compressive strength of concrete, determined from 6- by 12-in. control cylinders} \]
\[ f'' = \text{modulus of rupture of concrete, determined from 6- by 6- by 20-in. control beams} \]
\[ f_y = \text{yield strength of positive steel} \]
\[ f''_y = \text{yield strength of negative steel} \]
\[ v_c = \text{nominal unit shearing stress at diagonal tension cracking, equal to } \frac{V_c}{7/8 bd} \]

Constants, Parameters and Ratios

\[ c = \text{ratio of horizontal to vertical load} \]
\[ k_1 = \text{ratio of average compressive stress to maximum compressive stress in the concrete stress block} \]
\[ k_2 = \text{distance from the top of the horizontal member of a frame to the compressive force, divided by } k_d \]
\[ k_3 = \text{ratio of the maximum compressive stress in concrete stress block to cylinder strength, } f'_c \]
\[ p = \text{percentage of positive steel} \]
\[ p' = \text{percentage of negative steel} \]
4. Studies of Typical Culverts

The purpose of these studies was to obtain information regarding the range of the ratio of moment to axial load, and also the ratio of moment to shear, in order to design test specimens reasonably representative of typical culverts and, at the same time, to obtain basic information regarding the relative significance of the variables. The typical designs studied were those compiled by OCE and transmitted to us with the letter of 15 May 1956 (OVICS). These designs were for square culverts and for a ratio of horizontal to vertical load of one-third. The typical dimensions and steel areas used are summarized in Table 1.

From the data in Table 1, moment to axial load ratios were computed for two ratios of horizontal to vertical load, one-third and two-thirds, using the simplified expressions for the elastic moments given in Ref. (2) (See also First Progress Report). In these expressions it is assumed that:

(a) The horizontal load is uniformly distributed over the vertical member.

(b) Both vertical and horizontal members have the same thickness.

(c) The sections at the corners common to both horizontal and vertical members are infinitely stiff.

Table 2 shows the computed ratios of moment to axial load, in inches, for the typical culverts of Table 1.
5. **Variables Considered**

The general form of the specimens is that shown in Fig. 1. In arriving at their actual dimensions, the following variables were considered:

(a) concrete strength,
(b) section dimensions,
(c) steel percentage and continuity of negative steel,
(d) span length,
(e) horizontal to vertical load ratio.

6. **Variables Selected**

It was desirable to have a nominal concrete strength of about 4000 psi so that comparisons could be made with previous tests, (3), (4), (5), and (6). The same reasons led to the choice of a cross-section of 6-by-12-in.

The steel percentages were chosen so as to be approximately proportional to the moments. As can be seen from Table 2, the moments at mid-span are roughly twice those at column face for a horizontal to vertical load ratio of one-third. For a ratio of two-thirds, the positive and negative moments are approximately equal. The following steel percentages were used:

<table>
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<td>2%</td>
<td>1%</td>
</tr>
<tr>
<td>2/3</td>
<td>1%</td>
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There was reason to believe that the behavior of the member might depend on whether the negative steel was cut off at the positive moment region or made continuous throughout the member. For this reason,
it was considered necessary to have companion specimens, one with the negative steel continuous and the other with the steel cut off at the one-third points of the clear span, as this seemed to be the practice in the typical culverts studied. All other variables were the same in the companion specimens.

Of the three span lengths for the typical culverts studied, the two extreme cases (6- and 10-ft) were selected for testing. Having fixed the section dimensions and the clear span of the specimens, an analysis was made to determine the moment to axial load ratios which would be typical of those for the horizontal member of a square box culvert. The required length of leg of the test specimen (h in Fig. 1) was then easily determined by means of statics, for any given ratio of moment to axial load, and for a given ratio of horizontal to vertical load.

The ratio of horizontal to vertical load for actual culverts depends on the type of soil. It is believed that for most soils this ratio is between one-third and two-thirds. These two limiting cases were therefore selected for the test specimens.

Assuming the horizontal load to be uniformly distributed over the vertical member (a reasonable assumption for deep culverts), each of the horizontal members of a culvert is then acted upon by an axial load equal to one-half the horizontal load. Thus, the ratios of axial to vertical load for the test specimens are one-sixth and one-third for ratios of horizontal to vertical loads of one-third and two-thirds respectively.

7. Description of Test Specimens

Essentially four different types of specimens were used, a combination of two different span lengths and two ratios of horizontal to
vertical load. For each type, two specimens were made, one with the negative steel continuous throughout the horizontal member and the other with the negative steel cut off at the third-points of the clear span. No web reinforcement was provided in the horizontal member for these eight specimens. The general properties of the frames are listed in Table 3, and sketches are shown in Figs. 2 through 5. As can be seen from these sketches, stirrups were placed in the vertical members in order to avoid a possible failure there. In all cases, the centroid of the steel was at two inches from either the top or the bottom of the horizontal member.
III. MATERIALS AND FABRICATION PROCEDURES

8. Materials

(a) Cement - Marquette Type I portland cement was used in all frames. The cement was purchased in several lots from a local dealer.

(b) Aggregates - Wabash River sand and gravel, purchased from a local dealer were used in all frames. These aggregates are of a glacial origin. The coarse aggregate had a maximum size of about 1 in. with a fineness modulus of 6.3 to 7, and contained a rather high percentage of fines. The fineness modulus of the sand varied between 2.7 and 3.2. Both aggregates passed the usual specification tests. Absorption was about one percent by weight of surface dry aggregates.

(c) Concrete Mixes - Design of concrete mixes was based on results obtained previously for other investigations conducted in this laboratory using the same type of aggregates. The concrete was proportioned on the basis of a 3-in. slump. Table 4 lists the properties of the mixes. Compressive strengths are based on standard 6- by 12-in. cylinders. The modulus of rupture was determined by testing standard flexure specimens. Moisture samples were taken from the sand and gravel and the reported water-cement ratios are based on their results.

(d) Reinforcing Steel - Deformed bars were used in all frames. One coupon 2 ft long was cut from each bar and tested in tension. The bars used in each frame were matched as closely as possible according to their yield strengths, using bars cut from the same length whenever possible.
The value of the average yield point for the bars used in each frame is listed in Table 3. All bars were intermediate grade.

9. Fabrication and Curing of Specimens

All frames were cast in steel forms with adjustable end plates, with the legs extending upward. For mechanical strain gage readings to be made in the tests, 6-in. gage lines were marked on two outside bars, one of negative steel and one of positive steel, before the reinforcement was assembled and holes were punched and drilled. Corks of 1 3/8-in. diameter were attached with wire to the bars over each gage hole in order to provide access for strain measurements after casting.

The negative reinforcing steel was held in position by two or three chairs made from 1/4-in. mild steel bars. In order to hold the positive steel in position while casting, it was suspended by means of wires from transverse steel bars along the horizontal member. These steel bars, as well as the wires, were removed from the form as soon as casting was completed.

To facilitate handling, 1/4-in. steel hooks were imbedded in the concrete at the ends of the legs just after casting. The hooks were sawed off once the frame was ready to be placed in the testing rig.

Concrete was mixed from two to five minutes in a 6 cu ft capacity non-tilting drum type mixer. Two or three batches were required for each frame. In spite of the use of a butter mix to condition the mixer prior to the mixing of the first batch, the strength of two different batches of the same proportions varied to some extent. For the long frames, all of the third batch was usually placed in the legs and in all such cases the
reported strength of the frame is taken as the average of the first two batches corresponding to the horizontal portion of the specimen.

One 6- by 6- by 20-in. flexural control beam was cast from each batch in order to determine the modulus of rupture, and at least four 6- by 12-in. control cylinders were cast for the determination of compressive strength.

The concrete was placed in the form and in cylinder molds using a high frequency internal vibrator.

Several hours after casting, the top surface of the frames was trowelled smooth and all cylinders were capped with neat cement paste. The control specimens were removed from their forms the day after casting and placed near the frame. All concrete was cured under wet burlap for 5 days. The frames were removed from the forms one week after casting and stored with their control specimens in the laboratory until tested.
IV. TEST EQUIPMENT AND PROCEDURE

10. **Loading Apparatus**

A typical test setup is shown in Fig. 6. The uniform load was simulated by ten uniformly spaced concentrated loads. The spacings used for the two different span lengths are shown in Fig. 1. The ten jacks used are believed to produce a very reasonable approximation of a uniform load.

(a) **Vertical Loading Equipment** - Ten 10-ton Blackhawk hydraulic jacks were used, reacting against a steel beam attached to a frame anchored to the laboratory floor. The jacks were connected by high pressure hoses to a brass manifold, which in turn was connected to a 10,000-psi measuring gage and a hydraulic pump. The jacks were held with their bases against the reaction beam by two 1- by 1- by 1/8-in. angles clamped to the reaction beam.

The load was transmitted from the jack rams to the beam through 1.5-in. diameter chrome steel alloy balls, in order to maintain the loads vertically throughout the test. The balls rested in 1/8-in. depressions in the ends of the ram and in the center of a 6- by 6- by 3/4-in. loading plate. The frames were separated from the loading plate by 6- by 6- by 3/8-in. pieces of leather, to distribute the load uniformly. This arrangement facilitated the positioning of the jack and ball joint with respect to the frames.

Each frame was supported at the ends of the legs as shown in Fig. 6 by two support bearing blocks 12- by 6- by 1-in., attached with plaster to the ends of the legs. The support bearing block rested on a 4-in. diameter half-round at one end, and on a 2-in. diameter roller at the other. The roller and the half-round were both supported by 12- by 6- by 2-in. steel plates seated in plaster on concrete abutments.
The dial of the pressure gage was marked off in divisions of 100 psi of hydraulic pressure. Before use in the project, the gage was calibrated to read directly the load on the ten jacks. The calibration was again checked during the course of the tests and found not to have changed. The area of the jack rams was approximately 2 sq in. each, yielding a total capacity for the system of 200 kips. Because of the nature of the hydraulic system, the load on each jack was the same at any one time during the testing of a beam, regardless of the length of the ram extension, except for negligible differences in friction. Each jack was calibrated separately prior to use on the project and the differences between jacks were found to be negligible. The accuracy of the system was estimated to be about 0.4 kips total load.

(b) Axial Loading Equipment - The axial loading equipment was a completely separate unit, as can be seen in Figs. 6 and 7. It consisted of a hydraulic jack operating against one end of the frame, with the reaction to the jack supplied by tension rods acting against the other end of the frame. In order to allow the ends of the frames to rotate, a half-round rocker was supplied at one end, and at the other a 1.5-in. diameter chrome steel alloy ball was placed between two depressions in two steel plates, one acting against the beam through a leather pad, the other bearing directly the nuts of the tension rods. These two plates, as well as the one welded against the half-round rocker, were 6- by 12- by 1.5-in. The four tension rods used to connect the jack bearing plate to the plate acting at the other end of the frames were 7/8 in., threaded so that the system could be adjusted to accommodate frames of different lengths.

A Simplex 30-ton hydraulic jack was used to provide the axial load. It was connected by means of a hose to a gage and pump. The gage
and its corresponding pump were calibrated in a testing machine before testing. It was estimated that the load was measured within 0.2 kips.

11. Measuring Apparatus

(a) Deflections and Movements of the Legs - Three dial indicators reading to 0.001 in. were used to measure deflections at midspan and at the quarter-points of the clear span. They were mounted on a 2 1/2- by 2 1/2- by 1/4-in. angle seated on two steel saw horses imbedded in plaster on the floor.

Two other dial indicators, placed as shown in Fig. 6, were used to measure the movements of the legs.

(b) Steel Strains - Strains were measured in both the negative and positive steel for all frames. A Berry type mechanical gage with a sensitivity of approximately 0.00003 in. per in. per dial division was used on 6-in. gage lengths. The number of gage lines depended on the length of the frame, and varied from 19 to 42.

(c) Strains in the Concrete - Strains in the concrete were measured in the legs of all frames, chiefly to check the symmetry of the load, and at the column face sections on one side of all frames except F-1 and F-2. Three to five gage lines were used at each location.

A 10-in. Whittmore strain gage was used. Strains were estimated to the nearest millionth. Steel plugs 3/8 in. in diameter and 1/4 in. long with a gage hole drilled to a depth of about 1/8 in. were cemented to the concrete to establish the gage lines. Fig. 8 shows typical locations of gage lines for steel and concrete.
12. Testing Procedure

Special care was taken when placing the frame into the testing rig to insure that the axial load passed through the mid-vertical plane of the frame, and that the vertical load was actually vertical.

Once the frames were ready for loading, axial and vertical loads were both applied at the same time by simultaneous operation of the two pumps in order to keep the ratio of vertical to axial load constant, up to failure.

Testing was completed in ten to twenty increments of load up to final collapse, the load increments being smaller in the last stages of the test. The vertical load increments varied from 11 kips at the early stages to 2.2 kips in the last stages; the axial load was either one-third or one-sixth of the vertical load.

After each load increment, the valve between the pump and the jacks was closed. Deflection and strain readings were then taken and cracks observed and marked with ink. There was usually some drop-off in the load and some increase in deflection while readings were being taken. These changes were recorded before the next load increment was applied. Photographs of the test specimens were taken at important stages of the crack development and after failure. The control cylinders and beams were tested the same day the frame was tested.

A 5000-psi gage was being used for the axial load when testing frame F-5. When the gage capacity was reached before failure, at a load of about 85 percent of the maximum, it became necessary to remove the load in order to install a 10,000-psi gage. The load was completely removed in three increments and deflections for each increment were read. After the new gage was installed, three increments of load were applied to the frame
to again reach the previous load, and deflections were recorded after each load increment. No additional difficulties were encountered in the remainder of the tests.
V. BEHAVIOR AND MODE OF FAILURE

13. **Presentation of Test Data**

Test results are summarized in Table 5 and in Figs. 9 through 16, showing photographs of the first eight frames at both cracking and failure loads. For purposes of comparison, the computed flexural failure loads are also given in Table 5. They are obtained from the interaction diagrams in Figs. 17 and 18, in terms of axial load which was then converted to vertical load. Dead load has been neglected since it is never more than about 2 percent of the total load.

14. **Load-Deflection Curves**

Figs. 19 and 20 show the load-deflection curves for the frames without web reinforcement, the frames being grouped according to their span lengths. The decrease in load occurring while readings were being taken is not plotted. The decreases shown in Fig. 19 marked as $W_c$ correspond to the formation of the first major inclined crack. There are no such decreases in load in Fig. 20, since inclined cracking in the long-span frames was simultaneous with failure.

Before the appearance of the first flexural cracks, all frames behaved elastically, and deflections were proportional to loads. As the load increased, deflections were larger than those corresponding to a perfectly elastic material.

As can be seen from Figs. 19 and 20, the frames with the higher horizontal to vertical load ratio have a greater load-carrying capacity for a given deflection. No influence of the steel percentage was detected.
15. Distribution of Steel Strain

Two typical steel strain distributions, one for a 6-ft frame and the other for a 10-ft frame, are shown in Figs. 21 through 24 for both negative and positive reinforcement. The broken line on these figures represents the strains at ultimate load which were obtained by extrapolation. The accuracy of the extrapolated steel strain values at ultimate was checked by substituting them in the two equations from statics for the midspan section of all frames, using a value of \( \frac{k_p}{k} \) of 0.5. The measured values of ultimate load were checked within fifteen percent. Allowing some error because of the position of the lever arm, it is believed that the extrapolated values of steel strain at ultimate are within ten percent of the true values. The extrapolated values are not very accurate, especially for shear-compression failures, because of the sudden change in steel stress after the cracking load. The slope of the load-steel strain curves then becomes rather flat and the intersection with the ultimate load is not very precise.

After cracking, the steel stress increases suddenly at the critical section, as is clearly shown in Fig. 21. The same tendency was observed in the tests of simply-supported beams under concentrated loads (4). This characteristic is not observed in the cases where failure is in diagonal tension, as cracking and failure then occur simultaneously.

Steel strain distributions for negative and positive steel for frame F-7 are shown in Figs. 23 and 24. It can be seen that both the negative and positive steel had yielded before failure.

16. Concrete Strain

Fig. 25 shows the measured concrete strains at the column face sections for frame F-7 plotted versus total vertical load. The strains at
ultimate load are between 0.0008 and 0.0015 in. per in. At the north end, at which the failure occurred, the curve becomes rather flat, indicating that the maximum strain can be of the order of 0.002 in the outer fiber at failure. These strains are smaller than those corresponding to a flexural failure and are of the same order of magnitude as those obtained in tests of 6- by 12-in. control cylinders (Fig. 26).

The strains plotted in Fig. 25 are the average strains over a 10-in. gage length; the maximum local strain would be much higher. The strains shown in Fig. 26 were obtained using a gage length of 6 in.

No general statement can be made about the distribution of strain across the sections, as the gage length was too long. Also, on the tension side, most of the recorded strain is due to the concentrated effect of the cracks.

17. Behavior and Modes of Failure

Typical crack patterns for all frames without web reinforcement are shown in Figs. 9 through 16. Flexural cracks started in the regions of maximum moment, usually near midspan. For all frames tested so far, the positive moment at midspan was larger or at least equal to the moment at the column face. Flexural cracks formed at the first or second load increments, according to the pattern of principal stresses. As load increased, they extended up to a certain stage at which their progression stopped almost completely. After one or two load increments, the first indications of diagonal cracks appeared, small at first, and near the point of contraflexure at about mid-depth. The pattern of diagonal cracks was sometimes symmetrical; that is, similar cracks were formed simultaneously at both ends of the specimen. At other times, a crack formed at only one end of the specimen.
After the first diagonal cracks had formed, the subsequent behavior varied according to the mode of failure.

Shear failures will be considered in two general categories. Shear-compression failures, where the frame is able to carry additional load after formation of a fully developed diagonal crack, and diagonal tension failures, where the sudden appearance of a fully developed diagonal crack causes collapse of the frame. Within these two general types, a further subdivision can be made as to whether the specimen fails before, after, or simultaneously with the first yielding of the reinforcement.

All of the long-span frames without web reinforcement failed in diagonal tension, as defined above. In the short-span frames, however, the crack development was much more gradual, and the load at which a diagonal crack was considered to be fully developed could not be determined precisely by visual observation. Generally, it was considered as fully developed when the diagonal crack started progressing towards both the midspan and column face sections.

The load corresponding to this stage of the tests was called the cracking load. A diagonal tension failure can thus also be defined as a failure occurring suddenly at the cracking load.

In accordance with the above definitions, the frames tested without web reinforcement can be classified as follows:
<table>
<thead>
<tr>
<th>Frame No.</th>
<th>Ultimate load</th>
<th>Cracking load</th>
<th>Horiz. load</th>
<th>Vert. load</th>
<th>Mode of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-1</td>
<td>1.36</td>
<td>1/3</td>
<td></td>
<td></td>
<td>Clearly shear-compression failure before first yielding of the reinforcement. Cracking load reasonably well defined. Major diagonal cracks occurred at both ends of the frame simultaneously.</td>
</tr>
<tr>
<td>F-2</td>
<td>1.42</td>
<td>1/3</td>
<td></td>
<td></td>
<td>Clearly shear-compression failure before yielding of the reinforcement. Cracking load well defined with just one diagonal crack at the north end. Another symmetrically placed major crack at the south end at load increment before last. Final failure at north end.</td>
</tr>
<tr>
<td>F-5</td>
<td>1.07</td>
<td>2/3</td>
<td></td>
<td></td>
<td>Shear-compression failures at first yielding of midspan reinforcement. Unexplained differences in cracking (See Figs. 13 and 14). Even though steel reached yielding, the failure loads did not correspond to those of yielding because a major diagonal crack had formed previously.</td>
</tr>
<tr>
<td>F-6</td>
<td>1.25</td>
<td>2/3</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
18. Effect of Variables

As noted elsewhere (2), (4) and (7), the span length has a definite bearing on the mode of failure. Of the eight frames tested without web reinforcement, the four with a clear span of 6 ft failed in shear-compression at varying ratios of ultimate to cracking loads, while the frames with a clear span of 10 ft failed in diagonal tension.

The variation of the ratio of horizontal to vertical load introduced other differences in behavior and mode of failure. The first four frames tested, F-1 through F-4, all having a ratio of horizontal to vertical loads of one-third, failed at loads below those causing first yielding of the reinforcement.

Of the frames having a ratio of horizontal to vertical load of 2/3, the shorter ones, F-5 and F-6, failed at first yielding of the positive reinforcement with unexplained differences in cracking pattern. The longer
frames, F-7 and F-8, failed in diagonal tension after yielding of both the positive and negative steel.

No influence of the continuity of negative steel on the mode of failure was detected. The reason for this is probably that the point of contraflexure in all frames was well outside the cut-off point for the negative steel, and the carrying capacity of the midspan section is little affected by the presence of compressive steel, as shown in Fig. 17. However, cutting off the steel at one-third of the clear span might make a difference in behavior and mode of failure for other values of the ratio of positive to negative moment. This possibility needs further investigation.

Figs. 17 and 18 compare the measured cracking and failure loads of the frames tested with the corresponding computed flexural capacities. For a given section, load is applied at a constant ratio of moment to axial load, thus the critical sections for flexure are at midspan and at the column face since these sections have the largest absolute values of the ratio of moment to axial load.

Fig. 17 and 18 provide only approximate values of flexural capacities, since the interaction diagrams have been constructed on the basis of an average yield point stress of the steel of 48,000 psi, while the actual steel used had yield strengths varying from this value as noted in Table 3.

The interaction diagrams for yield load were constructed on the basis of the straight-line theory. Those for ultimate were based on the procedure outlined in Ref. (2).
It is shown in Fig. 18 that the failure loads for frames F-7 and F-8 can be predicted quite closely by interaction diagrams. Measured failure loads and those given by the interaction diagram are almost coincident.

In all other cases, flexural failure loads were above the actual shear failure loads of the frames.
The horizontal portion of a frame like those tested can be regarded as a member acted upon by uniform and axial loads and by restraining moments at the ends. This type of loading produces a varying shear diagram and a varying moment diagram. Both the positive and negative moment regions must be considered, as illustrated in Fig. 27, where shear and moment diagrams are shown for frame F-8.

Since the ratio of horizontal to vertical load is constant throughout the test, the ratio of moment to axial load and the ratio of shear to axial load are independent of the load level and are thus known, but they vary for every section along the horizontal member.

If a limiting shear or moment criterion is to be used to determine the capacity of the frames, it is then very important to know the location of the critical section; that is, the section at which the frame fails. If, on the other hand, the capacity of the frames is to be determined by means of known steel stresses, the position of the critical section is not so important.

The quantities to be measured in the tests were so chosen that either of the two approaches could be followed.

An attempt was made to correlate the ultimate capacities of the frames tested on the basis of the existing analyses and empirical expressions (3), (4) and (5), but no definite trend was found. Actually, the existing empirical relationships were based on tests that do not correspond exactly with those reported here. Moreover, if a correlation on the basis of
ultimate loads is to be made, then the values of ultimate loads for frames failing in shear-compression and for frames failing in diagonal tension cannot be compared on the same basis.

It is believed that since frames undergo considerable damage at diagonal tension cracking, and since shear-compression strength is not as predictable as diagonal tension strength, analysis and design procedures should be based on diagonal tension cracking. An analysis on this basis should then take care of both diagonal tension and shear-compression failures.

20. Nominal Shearing Stress and Location of Critical Section at Cracking Load

In correlating the test data at cracking load, it was found from the plot in Fig. 28 of the axial load at cracking versus the shear at cracking at the critical section, that the plotted points could be closely represented by the following equation:

\[ v_c = 0.591 \frac{N_c}{7/8 \, bd} + 0.175 \frac{f'_c}{1 + 0.85 \frac{f'_c}{1000}} \]  

(1)

where \( v_c \) is the nominal shearing unit stress, equal to \( \frac{V_c}{7/8 \, bd} \).

Although the compressive strength was not a variable, it was felt that its effect could be satisfactorily represented by the factor developed by Bernaert (3), on the basis of simply-supported uniformly loaded beams failing in shear. This factor has been used to modify both the abscissas and ordinates in the plot of Fig. 28, from which equation (1) was obtained.

Equation (1) and the measured values are plotted in Fig. 28. Table 6 shows a comparison between measured and computed values. It is
seen that the average error for the frames without web reinforcement is 5 percent, with a range of -6 to +8 percent.

Included also in Table 6 and Fig. 28 are the values for frames F-9, F-10 and F-11 with web reinforcement, which will be discussed in Chapter VII of this report.

Equation (1) can be used only if the position of the critical section is known in advance, so that the ratio of shear to axial load can be determined.

In Table 7 and Fig. 29 the measured values of the ratio of shear to axial load at the critical section were correlated with the ratio of mid-span moment to the total static moment between the column face sections; that is, with the degree of fixity of the horizontal member, which depends only on the loading conditions. Table 7 and Fig. 29 include also values for the frames with web reinforcement. They will be discussed in Chapter VII.

In Figs. 28 and 29 two independent relationships between cracking and axial load at the critical section are given, in function of known quantities. Their usefulness and limitations will be discussed in the following section.

21. Effect of Variables

In Fig. 29, apart from the degree of fixity, two different influences can be observed on the ratio of shear to axial load at cracking: the span length and the ratio of horizontal to vertical load. Whether the ratio of shear to axial load depends on only one of them or on a combination of both cannot yet be said, since the range of the degrees of fixity used is
too limited. Further tests need to be performed with extreme values of the ratio $M_{t/2}/M_{st}$.

With the exception of frame F-6, the critical section was always assumed to be that at which the curved portion of the major diagonal crack crossed mid-depth of the frame.

In general, it can be stated that the major diagonal cracks tended to form near the point of contraflexure, at about mid-depth, and this introduced uncertainties in the analysis, since the moment varies considerably with a small change in the position of the critical section, as shown in Fig. 27.

Cracking and failure loads are shown in Table 5. It can be seen that, for a given span length, the load-carrying capacity is not greatly increased by a comparatively large increase in axial load.

The effect of changing the steel percentage from one to two percent at the positive moment region was also rather small. A more specific statement about the effect of the steel percentage cannot be made in view of the limited number of tests, with only two different steel percentages.
VII. FRAMES WITH WEB REINFORCEMENT

22. **Design**

Since the first eight frames were tested without any kind of web reinforcement in the horizontal member, it was considered desirable to determine whether a small amount of web reinforcement in the form of bent bars would have an appreciable effect on the load-carrying capacity, the position of the critical section, the cracking pattern, or the mode of failure.

After studying several possible arrangements of bent bars, and taking into consideration the limited width of the specimens, it was decided to provide a single bent bar at each end of the specimen, symmetrically placed about midspan and inclined at 45 degrees.

Three specimens were designed: F-9, F-10 and F-11, similar to F-6, F-2 and F-4 respectively, except for the presence of the bent bars at approximately the critical sections, and for unintentional differences in concrete strength. Figs. 30 and 31 show the details of these frames.

These specimens were cast, cured, and tested in exactly the same manner as their companion specimens without web reinforcement, as described in Chapters III and IV of this report.

23. **Behavior and Mode of Failure**

Test results for the frames with web reinforcement are summarized in Table 5 and in Figs. 32 through 34, where photographs are shown at cracking and failure loads. Load versus midspan deflection curves for these frames are shown in Fig. 35.
The behavior of the frames with and without web reinforcement was similar up to cracking load. However, after cracking load the following differences were noted:

(a) As stated before, the bent bar was located at the section corresponding to the critical section of the companion frame. The major diagonal crack for the frames with web reinforcement was shifted towards the end of the frame, but always in the vicinity of the section crossed by the bent bar. This can be seen in Figs. 32 through 34 where the bent bar is shown as a dotted line outside the frame.

(b) The slope of the major diagonal crack was flatter.

(c) The crack development was slower, and more small diagonal cracks were formed in the vicinity of the major diagonal crack.

(d) The deflections were somewhat smaller.

The different modes of failure for the frames without web reinforcement can be summarized as follows:

**Frame F-9** - Clear span 6 ft. Horizontal to vertical load ratio \(2/3\). Diagonal tension failure after yielding of the midspan reinforcement. No appreciable diagonal cracks other than at failure load. In this case, the presence of web reinforcement reversed the mode of failure from shear-compression for its companion specimens (F-5 to F-6) to diagonal tension. The critical section was at the same location.

**Frame F-10** - Clear span 6 ft. Horizontal to vertical load ratio \(1/3\). Shear-compression failure at first yielding of both positive and negative reinforcement. Cracking load for this frame was the same as for F-2, its companion specimen. The critical section was in almost the same location. Failure to cracking load ratio, 2.07.
Frame F-11 - Clear span 10 ft. Horizontal to vertical load ratio 1/3. Shear-compression failure almost at yielding of the negative steel. Cracking load in this case was the failure load of its companion specimens F-3 and F-4, thus the presence of web reinforcement changed the mode of failure from diagonal tension to shear compression. The critical section moved 17 in. towards the end of the frame. Failure to cracking load ratio 1.23.

As can be observed in Fig. 33, there was some crushing in the outside corner of the south side of frame F-10, at failure. It is believed that this crushing was caused by compression developed in the bend of the negative steel, and that, for an actual culvert where the concrete is confined laterally, there would be no such possibility.

24. Comparison With Previous Correlation

As can be seen in Tables 6 and 7, equation (1) approximates within 9 percent the values of the shearing stress for frames F-9 and F-10, but is 45 percent off for frame F-11. This is due to the fact that the critical section did not greatly change for the first two frames, while it moved 17 in. for frame F-11. If the critical section is taken where the bent bar was located, then equation (1) holds. Actually the first diagonal crack started to form at that location, but was too small to be considered a major diagonal crack.

25. General Remarks

The values in Table 5 show that the load carrying capacity was always greater in the frames with web reinforcement. Part of this increase in load carrying capacity was undoubtedly due to a larger value of concrete strength; however, the influence of a properly located bent bar is clearly shown.
The amount of web reinforcement was not sufficient to prevent a flexural failure. However, the necessary amount and location of web reinforcement to insure a flexural failure was not expected to be determined from these tests which were exploratory in nature and of very limited scope.

Nevertheless, the results of these three tests suggest that a relatively small amount of web reinforcement might be sufficient to prevent shear failures if the critical sections could be located with reasonable accuracy.
VIII. SUMMARY

The object of this investigation was to explore the behavior and strength in shear of uniformly-loaded reinforced concrete frames.

Eleven frames were tested, all under-reinforced in the horizontal member. Eight frames had no web reinforcement, while three had web reinforcement in the horizontal member in the form of bent bars symmetrically placed about midspan.

Two different clear spans were used, six and ten feet; and two different ratios of horizontal to vertical load, one-third and two-thirds. The reinforcement was provided so as to be approximately in proportion to the maximum positive and negative moments thus produced, using typical ratios of moment to axial load.

Of the eight frames without web reinforcement tested, the four with longer span failed in diagonal tension, and the four with shorter spans failed in shear-compression. For a given span length, the load carrying capacity was not greatly increased with increasing axial load.

Using the test data, a relationship was found between the shear at the critical section and the axial load, both at cracking load. The ratio of axial load to shear at cracking was related to another independent parameter, the degree of fixity.

The ratio of axial load to shear at cracking at the critical section seemed to depend on the span length and on the ratio of horizontal to vertical load, in addition to the degree of fixity. However, the effects of these additional variables could not be determined precisely from the limited number of tests made so far.
After testing the frames without web reinforcement, it seemed desirable to see whether a small amount of web reinforcement in the form of properly located bent-up bars would have an appreciable effect on cracking pattern, mode of failure, location of the critical section and load carrying capacity.

On this basis three frames with web reinforcement were designed, cast and tested. The web reinforcement was not enough to prevent shear failure, but had some influence on the load carrying capacity. The cracking pattern was not greatly changed except for frame F-11 where the critical section moved about 17 in. towards the end of the frame. The modes of failure for frames F-9 and F-11 were different from those of their companion specimens without web reinforcement.
REFERENCES


### TABLE 1

**DIMENSIONS OF TYPICAL CULVERT SECTIONS**

<table>
<thead>
<tr>
<th>$l$ (ft)</th>
<th>$T_1$ (in.)</th>
<th>$T_2$ (in.)</th>
<th>$S_1$</th>
<th>$S_2$.</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>13</td>
<td>10</td>
<td># 7 at 6&quot;</td>
<td># 6 at 10&quot;</td>
</tr>
<tr>
<td>6</td>
<td>16.5</td>
<td>11</td>
<td>8 at 7</td>
<td>7 at 12</td>
</tr>
<tr>
<td>6</td>
<td>19</td>
<td>12.5</td>
<td>8 at 6</td>
<td>7 at 11</td>
</tr>
<tr>
<td>6</td>
<td>21</td>
<td>14</td>
<td>9 at 7</td>
<td>7 at 10</td>
</tr>
<tr>
<td>6</td>
<td>22.5</td>
<td>15</td>
<td>9 at 6</td>
<td>7 at 9</td>
</tr>
<tr>
<td>8</td>
<td>17</td>
<td>11.5</td>
<td>8 at 6-1/2</td>
<td>7 at 11</td>
</tr>
<tr>
<td>8</td>
<td>21</td>
<td>14</td>
<td>9 at 6-1/2</td>
<td>7 at 9</td>
</tr>
<tr>
<td>8</td>
<td>24.5</td>
<td>16.5</td>
<td>10 at 7-1/2</td>
<td>7 at 8</td>
</tr>
<tr>
<td>8</td>
<td>27</td>
<td>18</td>
<td>10 at 6-1/2</td>
<td>8 at 9-1/2</td>
</tr>
<tr>
<td>8</td>
<td>29</td>
<td>19.5</td>
<td>10 at 6</td>
<td>8 at 8-1/2</td>
</tr>
<tr>
<td>10</td>
<td>20.5</td>
<td>14</td>
<td>10 at 8</td>
<td>8 at 10-1/2</td>
</tr>
<tr>
<td>10</td>
<td>25.5</td>
<td>17</td>
<td>10 at 6-1/2</td>
<td>8 at 9</td>
</tr>
<tr>
<td>10</td>
<td>30</td>
<td>20</td>
<td>10 at 6</td>
<td>8 at 8-1/2</td>
</tr>
<tr>
<td>10</td>
<td>33.5</td>
<td>22.5</td>
<td>11 at 6-1/2</td>
<td>8 at 8-1/2</td>
</tr>
<tr>
<td>10</td>
<td>36</td>
<td>24</td>
<td>11 at 6</td>
<td>8 at 7</td>
</tr>
</tbody>
</table>

[Diagram of a typical culvert section with labels for $l$, $T_1$, $S_1$, $S_2$, and $T_2$.]
TABLE 2

VALUES OF M/N FOR CULVERTS OF TABLE 1

<table>
<thead>
<tr>
<th>l</th>
<th>AT MIDSPAN</th>
<th>AT COLUMN FACE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>c = 1/3</td>
<td>c = 2/3</td>
</tr>
<tr>
<td>ft</td>
<td>in.</td>
<td>in.</td>
</tr>
<tr>
<td>----</td>
<td>------------</td>
<td>----------------</td>
</tr>
<tr>
<td>6</td>
<td>25</td>
<td>10</td>
</tr>
<tr>
<td>6</td>
<td>24</td>
<td>9</td>
</tr>
<tr>
<td>6</td>
<td>24</td>
<td>9</td>
</tr>
<tr>
<td>6</td>
<td>23</td>
<td>9</td>
</tr>
<tr>
<td>8</td>
<td>34</td>
<td>13</td>
</tr>
<tr>
<td>8</td>
<td>34</td>
<td>12</td>
</tr>
<tr>
<td>8</td>
<td>32</td>
<td>12</td>
</tr>
<tr>
<td>8</td>
<td>31</td>
<td>12</td>
</tr>
<tr>
<td>10</td>
<td>42</td>
<td>16</td>
</tr>
<tr>
<td>10</td>
<td>40</td>
<td>15</td>
</tr>
<tr>
<td>10</td>
<td>39</td>
<td>15</td>
</tr>
<tr>
<td>10</td>
<td>39</td>
<td>15</td>
</tr>
<tr>
<td>10</td>
<td>38</td>
<td>15</td>
</tr>
</tbody>
</table>
### TABLE 3

PROPERTIES OF THE FRAMES
For All Frames \( b = 6 \) in., \( d = 10 \) in.

<table>
<thead>
<tr>
<th>Spec.</th>
<th>Clear Span ft</th>
<th>( h ) in.</th>
<th>( f'_c ) psi</th>
<th>( f_y ) ksi</th>
<th>( f'_y ) ksi</th>
<th>( p ) %</th>
<th>( p' ) %</th>
<th>( c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-1</td>
<td>6</td>
<td>27</td>
<td>4100</td>
<td>48.0</td>
<td>48.0</td>
<td>2</td>
<td>1</td>
<td>1/3</td>
</tr>
<tr>
<td>*F-2</td>
<td>6</td>
<td>27</td>
<td>5000</td>
<td>47.6</td>
<td>47.6</td>
<td>2</td>
<td>1</td>
<td>1/3</td>
</tr>
<tr>
<td>F-3</td>
<td>10</td>
<td>43</td>
<td>4130</td>
<td>47.9</td>
<td>47.1</td>
<td>2</td>
<td>1</td>
<td>1/3</td>
</tr>
<tr>
<td>*F-4</td>
<td>10</td>
<td>43</td>
<td>3750</td>
<td>42.4</td>
<td>45.6</td>
<td>2</td>
<td>1</td>
<td>1/3</td>
</tr>
<tr>
<td>F-5</td>
<td>6</td>
<td>17</td>
<td>3500</td>
<td>45.8</td>
<td>45.8</td>
<td>1</td>
<td>1</td>
<td>2/3</td>
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<tr>
<td>*F-6</td>
<td>6</td>
<td>17</td>
<td>3400</td>
<td>52.3</td>
<td>52.3</td>
<td>1</td>
<td>1</td>
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<td>F-7</td>
<td>10</td>
<td>27</td>
<td>4000</td>
<td>48.0</td>
<td>44.9</td>
<td>1</td>
<td>1</td>
<td>2/3</td>
</tr>
<tr>
<td>*F-8</td>
<td>10</td>
<td>27</td>
<td>5200</td>
<td>47.9</td>
<td>47.9</td>
<td>1</td>
<td>1</td>
<td>2/3</td>
</tr>
<tr>
<td>**F-9</td>
<td>6</td>
<td>17</td>
<td>5700</td>
<td>49.7</td>
<td>49.7</td>
<td>1</td>
<td>1</td>
<td>2/3</td>
</tr>
<tr>
<td>**F-10</td>
<td>6</td>
<td>27</td>
<td>4090</td>
<td>47.8</td>
<td>48.4</td>
<td>2</td>
<td>1</td>
<td>1/3</td>
</tr>
<tr>
<td>**F-11</td>
<td>10</td>
<td>43</td>
<td>5000</td>
<td>45.8</td>
<td>47.2</td>
<td>2</td>
<td>1</td>
<td>1/3</td>
</tr>
</tbody>
</table>

Notes:

(a) Specimen numbers preceded by * and ** had the negative steel cut-off at approximately one-third of the clear span.

(b) Specimen numbers preceded by ** had web reinforcement.

(c) \( f'_y \) and \( p' \) are the yield point stress and steel percentage, respectively, of the negative steel.

(d) \( f'_c \) values are average from standard control cylinders tested the same day the test was performed. For the long specimens one whole batch of concrete was necessary for just the legs. Control cylinders from that batch are not taken into consideration.
### TABLE 4

**CONCRETE MIXES**

<table>
<thead>
<tr>
<th>Frame No.</th>
<th>Batch</th>
<th>Cement:Sand:Gravel by Weight</th>
<th>Cement/Water by Weight</th>
<th>Slump in.</th>
<th>Compressive Strength, $f'_c$ psi</th>
<th>Modules of Rupture, $f'_r$ psi</th>
<th>Age at Test Days</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>F-1</strong></td>
<td>1</td>
<td>1.00:3.42:5.07</td>
<td>1.28</td>
<td>7</td>
<td>4080</td>
<td>500</td>
<td>48</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.00:3.42:5.07</td>
<td>1.40</td>
<td>6</td>
<td>4250</td>
<td>458</td>
<td></td>
</tr>
<tr>
<td><strong>F-2</strong></td>
<td>1</td>
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<td>56</td>
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<tr>
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<td>1.44</td>
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<tr>
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<td>2</td>
<td>1.00:3.42:5.06</td>
<td>1.29</td>
<td>5</td>
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<td>1.00:3.42:5.06</td>
<td>1.26</td>
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<td>2</td>
<td>1.00:3.46:5.05</td>
<td>1.44</td>
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<tr>
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<td>3</td>
<td>1.00:3.46:5.05</td>
<td>1.44</td>
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<td>Batch</td>
<td>Cement:Sand:Gravel by Weight</td>
<td>Cement/Water by Weight</td>
<td>Slump in.</td>
<td>Compressive Strength, $f'_c$ psi</td>
<td>Modules of Rupture, $f'_{r}$ psi</td>
<td>Age at Test Days</td>
</tr>
<tr>
<td>-----------</td>
<td>-------</td>
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<td>------------------------</td>
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<td>1.61</td>
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<td>3850</td>
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<tr>
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<td>3</td>
<td>1.00:3.36:5.01</td>
<td>1.61</td>
<td>4 1/2</td>
<td>3730</td>
<td>533</td>
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<tr>
<td>F-8</td>
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<td>1.00:3.42:5.01</td>
<td>1.65</td>
<td>1/2</td>
<td>5330</td>
<td>592</td>
<td>42</td>
</tr>
<tr>
<td></td>
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<td>1.00:3.42:5.01</td>
<td>1.61</td>
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</tr>
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<td>1.00:3.42:5.01</td>
<td>1.61</td>
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<td>5350</td>
<td>592</td>
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<td>F-9</td>
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<td>1/2</td>
<td>5360</td>
<td>467</td>
<td>29</td>
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<tr>
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<td>2</td>
<td>1.00:3.53:5.15</td>
<td>1.44</td>
<td>3 1/2</td>
<td>4830</td>
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<tr>
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<td>3</td>
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### Table 5

**TEST RESULTS**

<table>
<thead>
<tr>
<th>Frame No.</th>
<th>Clear Span (ft)</th>
<th>h (in)</th>
<th>Load at Cracking</th>
<th>Load at Failure</th>
<th>Theor. Vert. Load at Flexural Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Vert. Axial</td>
<td>Vert. Axial</td>
<td>Yielding Ultimate</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>kips</td>
<td>kips</td>
<td>kips kips</td>
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</tbody>
</table>

#### (a) Frames without web reinforcement

<table>
<thead>
<tr>
<th>Frame</th>
<th>6</th>
<th>27</th>
<th>61.6</th>
<th>9.3</th>
<th>83.6</th>
<th>12.7</th>
<th>159.0</th>
<th>174.0</th>
<th>SC</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-2</td>
<td>6</td>
<td>27</td>
<td>66.0</td>
<td>11.0</td>
<td>93.5</td>
<td>14.3</td>
<td>159.0</td>
<td>174.0</td>
<td>SC</td>
</tr>
<tr>
<td>F-3</td>
<td>10</td>
<td>43</td>
<td>57.2</td>
<td>9.5</td>
<td>57.2</td>
<td>9.5</td>
<td>63.0</td>
<td>69.0</td>
<td>DT</td>
</tr>
<tr>
<td>F-4</td>
<td>10</td>
<td>43</td>
<td>51.7</td>
<td>8.8</td>
<td>51.7</td>
<td>8.8</td>
<td>63.0</td>
<td>69.0</td>
<td>DT</td>
</tr>
<tr>
<td>F-5</td>
<td>6</td>
<td>17</td>
<td>90.3</td>
<td>27.0</td>
<td>96.8</td>
<td>30.5</td>
<td>126.0</td>
<td>135.9</td>
<td>SC</td>
</tr>
<tr>
<td>F-6</td>
<td>6</td>
<td>17</td>
<td>88.0</td>
<td>27.4</td>
<td>110.0</td>
<td>34.9</td>
<td>126.0</td>
<td>135.9</td>
<td>SC</td>
</tr>
<tr>
<td>F-7</td>
<td>10</td>
<td>27</td>
<td>60.5</td>
<td>20.8</td>
<td>60.5</td>
<td>20.8</td>
<td>57.0</td>
<td>63.0</td>
<td>DT</td>
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<tr>
<td>F-8</td>
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<td>27</td>
<td>57.2</td>
<td>19.1</td>
<td>57.2</td>
<td>19.1</td>
<td>50.1</td>
<td>57.0</td>
<td>DT</td>
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#### (b) Frames with web reinforcement

<table>
<thead>
<tr>
<th>Frame</th>
<th>6</th>
<th>17</th>
<th>136.5</th>
<th>45.5</th>
<th>136.5</th>
<th>45.5</th>
<th>DT</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-9</td>
<td>6</td>
<td>27</td>
<td>66.0</td>
<td>11.0</td>
<td>136.4</td>
<td>22.8</td>
<td>SC</td>
</tr>
<tr>
<td>F-10</td>
<td>6</td>
<td>43</td>
<td>57.2</td>
<td>9.5</td>
<td>70.4</td>
<td>11.7</td>
<td>SC</td>
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</tbody>
</table>

**Notes:**

*(a) SC - Shear-compression failure
   DT - Diagonal tension failure

(b) F-9, F-10 and F-11 are companion specimens of F-6, F-2 and F-4, respectively.

(c) Values include live load only.
TABLE 6

COMPARISON OF MEASURED AND COMPUTED VALUES
OF NOMINAL UNIT SHEARING STRESS AT DIAGONAL TENSION CRACKING
Measured Values Include Live Load Only

<table>
<thead>
<tr>
<th>Frame No.</th>
<th>Measured $v_c$ (psi)</th>
<th>Computed* $v_c$ (psi)</th>
<th>Ratio $\frac{Measured}{Computed}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-1</td>
<td>286</td>
<td>265</td>
<td>1.08</td>
</tr>
<tr>
<td>F-2</td>
<td>276</td>
<td>291</td>
<td>0.95</td>
</tr>
<tr>
<td>F-3</td>
<td>267</td>
<td>268</td>
<td>1.00</td>
</tr>
<tr>
<td>F-4</td>
<td>238</td>
<td>255</td>
<td>0.93</td>
</tr>
<tr>
<td>F-5</td>
<td>476</td>
<td>458</td>
<td>1.04</td>
</tr>
<tr>
<td>F-6</td>
<td>476</td>
<td>462</td>
<td>1.03</td>
</tr>
<tr>
<td>F-7</td>
<td>419</td>
<td>393</td>
<td>1.07</td>
</tr>
<tr>
<td>F-8</td>
<td>462</td>
<td>383</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Average 1.01</td>
<td>Average error .05</td>
</tr>
<tr>
<td>F-9</td>
<td>743</td>
<td>680</td>
<td>1.09</td>
</tr>
<tr>
<td>F-10</td>
<td>303</td>
<td>284</td>
<td>1.06</td>
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<tr>
<td>F-11</td>
<td>400</td>
<td>276</td>
<td>1.45</td>
</tr>
</tbody>
</table>

* Computed values using equation (1).
TABLE 7

CRACKING SHEAR AND AXIAL LOAD AT CRITICAL SECTION

<table>
<thead>
<tr>
<th>Frame No.</th>
<th>x (in.)</th>
<th>Axial Load N_c (kips)</th>
<th>Shear V_c (kips)</th>
<th>V_c / N_c</th>
<th>M_l/2 / M_{st}</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-1</td>
<td>25</td>
<td>9.3</td>
<td>15.0</td>
<td>1.61</td>
<td>.731</td>
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<tr>
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<td>27</td>
<td>11.0</td>
<td>14.5</td>
<td>1.32</td>
<td>.667</td>
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<tr>
<td>F-3</td>
<td>36</td>
<td>9.5</td>
<td>14.0</td>
<td>1.47</td>
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<tr>
<td>F-4</td>
<td>37</td>
<td>8.8</td>
<td>12.5</td>
<td>1.42</td>
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<tr>
<td>F-5</td>
<td>21</td>
<td>27.0</td>
<td>25.0</td>
<td>0.93</td>
<td>.580</td>
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<tr>
<td>F-6</td>
<td>21*</td>
<td>27.4</td>
<td>25.0</td>
<td>0.91</td>
<td>.549</td>
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<td>1.06</td>
<td>.456</td>
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<td>19.1</td>
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<td>1.01</td>
<td>.478</td>
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<td>F-9 (F-6)**</td>
<td>21</td>
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<td>39.0</td>
<td>0.86</td>
<td>.494</td>
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<td>F-10 (F-2)</td>
<td>26</td>
<td>11.0</td>
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<td>F-11 (F-4)</td>
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<td>9.5</td>
<td>21.0</td>
<td>2.21</td>
<td>.633</td>
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</tbody>
</table>

Notes:

* The critical location for frame F-6 was taken like that of F-5 because of unexplained differences in cracking pattern.

** Frame numbers in parentheses are those of the corresponding frames without web reinforcement.

*** The static moment is taken as the total moment between column face sections.
FIG. 1 SHAPE OF TEST SPECIMEN AND SPACING OF LOADS
FIG. 2 DETAILS OF FRAMES F-1 AND F-2
FIG. 3 DETAILS OF FRAMES F-3 AND F-4
FIG. 4  DETAILS OF FRAMES F-5 AND F-6
FIG. 5 DETAILS OF FRAMES F-7 AND F-8
FIG. 8 TYPICAL GAGE LINE LOCATIONS
Vertically Axial

Cracking Load

Vertical  61.6 kips
Axial     9.3 kips

Ultimate Load

Vertical  83.6 kips
Axial     12.7 kips

DATA

L = 1/3
L = 6 ft
h = 27 in.
f'c = 4100 psi
f'y = 48.0 ksi
f'v = 48.0 ksi
p = 2 per cent
p' = 1 per cent

FIG. 9 FRAME F-1
Failure in shear - compression
Cracking Load

Vertical  66.0 kips
Axial     11.0 kips

Ultimate Load

Vertical  93.5 kips
Axial     14.3 kips

DATA

\[ c = \frac{1}{3} \]
\[ l = 6 \text{ ft} \]
\[ h = 27 \text{ in.} \]
\[ f_c' = 5000 \text{ psi} \]
\[ f_y = 47.6 \text{ ksi} \]
\[ f_y' = 47.6 \text{ ksi} \]
\[ p = 2 \text{ per cent} \]
\[ p' = 1 \text{ per cent} \]

FIG. 10 FRAME F-2
Failure in shear - compression
Ultimate Load

Vertical  57.2 kips
Axial     9.5 kips

FIG. 11 FRAME F-3
Failure in diagonal tension

DATA

\[ c = \frac{1}{3} \]
\[ l = 10 \text{ ft} \]
\[ h = 43 \text{ in.} \]
\[ f'_c = 4130 \text{ psi} \]
\[ f_y = 47.9 \text{ ksi} \]
\[ f'_y = 47.1 \text{ ksi} \]
\[ p = 2 \text{ per cent} \]
\[ p' = 1 \text{ per cent} \]
Ultimate Load

Vertical  51.7 kips
Axial     8.8 kips

FIG. 12 FRAME F-4
Failure in diagonal tension

DATA
\[ c = \frac{1}{3} \]
\[ l = 10 \text{ ft} \]
\[ h = 43 \text{ in.} \]
\[ f'_{c} = 3730 \text{ psi} \]
\[ f_{y} = 42.4 \text{ ksi} \]
\[ f'_{y} = 45.6 \text{ ksi} \]
\[ p = 2 \text{ per cent} \]
\[ p' = 1 \text{ per cent} \]
Cracking Load

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
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</thead>
<tbody>
<tr>
<td>Vertical</td>
<td>90.3 kips</td>
</tr>
<tr>
<td>Axial</td>
<td>27.0 kips</td>
</tr>
</tbody>
</table>

Ultimate Load

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
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</thead>
<tbody>
<tr>
<td>Vertical</td>
<td>96.8 kips</td>
</tr>
<tr>
<td>Axial</td>
<td>30.5 kips</td>
</tr>
</tbody>
</table>

DATA

c = 2/3
l = 6 ft
h = 17 in.
f'_c = 3500 psi
f'_y = 45.8 ksi
f'_y = 45.8 ksi
p = 1 per cent
p' = 1 per cent

FIG. 13 FRAME F-5
Failure in shear - compression
Cracking Load

Vertical  88.0 kips
Axial     27.4 kips

Ultimate Load

Vertical  110.0 kips
Axial     34.9 kips

DATA

$c = \frac{2}{3}$
$l = 6 \text{ ft}$
$h = 17 \text{ in.}$
$f'_c = 3400 \text{ psi}$
$f_y = 52.3 \text{ ksi}$
$f'_y = 52.3 \text{ ksi}$
$p = 1 \text{ per cent}$
$p = 1 \text{ per cent}$

FIG. 14  FRAME F-6
Failure in shear - compression
Ultimate Load

Vertical 60.5 kips
Axial 20.8 kips

DATA
\[ c = \frac{2}{3} \]
\[ l = 10 \text{ ft} \]
\[ h = 27 \text{ in.} \]
\[ f'_{c} = 4000 \text{ psi} \]
\[ f_{y} = 48.0 \text{ ksi} \]
\[ f'_{y} = 44.9 \text{ ksi} \]
\[ p = 1 \text{ per cent} \]
\[ p' = 1 \text{ per cent} \]

FIG. 15 FRAME F-7
Failure in diagonal tension
Ultimate Load

Vertical  57.2 kips
Axial    19.1 kips

DATA

\[ c = \frac{2}{3} \]
\[ l = 10 \text{ ft} \]
\[ h = 17 \text{ in.} \]
\[ f'_c = 5200 \text{ psi} \]
\[ f_y = 47.9 \text{ ksi} \]
\[ f'_y = 47.9 \text{ ksi} \]
\[ p = 1 \text{ per cent} \]
\[ p' = 1 \text{ per cent} \]

FIG. 16 FRAME F-8
Failure in diagonal tension
FIG. 17 COMPARISON BETWEEN THEORETICAL FLEXURAL CAPACITIES AND ACTUAL SHEAR FAILURE AXIAL LOADS. $c = 1/3$
FIG. 18 COMPARISON BETWEEN THEORETICAL FLEXURAL CAPACITIES AND ACTUAL SHEAR FAILURE AXIAL LOADS. $c = \frac{2}{3}$
FIG. 19 LOAD-DEFLECTION CURVES FOR 6-ft FRAMES
FIG. 20  LOAD-DEFLECTION CURVES FOR 10-ft FRAMES
FIG. 21 STEEL STRAIN DISTRIBUTION FOR FRAME F-1 (positive steel)
FIG. 22  STEEL STRAIN DISTRIBUTION FOR FRAME F-1
(negative steel)
FIG. 23  STEEL STRAIN DISTRIBUTION FOR FRAME F-7
(positive steel)
FIG. 24 STEEL STRAIN DISTRIBUTION FOR FRAME F-7
(negative steel)
FIG. 25 CONCRETE STRAINS VERSUS LOAD FOR FRAME F-7
FIG. 26  TYPICAL STRESS STRAIN CURVE FOR CONCRETE
FIG. 27  SHEAR AND MOMENT DIAGRAMS FOR FRAME F-8
FIG. 28 RELATIONSHIP BETWEEN CRACKING SHEAR AND AXIAL LOAD AT THE CRITICAL SECTION
FIG. 29 INFLUENCE OF SPAN LENGTH AND RATIO OF HORIZONTAL TO VERTICAL LOAD ON LOCATION OF CRITICAL SECTION
FIG. 30 DETAILS OF FRAMES F-9 AND F-10

FRAME F-9

FRAME F-10

1 - #4 bar
2 - #4 bars

2 - #4 bars
1 - #4 bar

23 1/4"
2 - #7 bars

8"
FIG. 31 DETAILS OF FRAME F-11
Ultimate Load

Vertical  136.5 kips
Axial     45.5 kips

FIG. 32 FRAME F-9
Failure in diagonal tension

DATA

\[ c = \frac{2}{3} \]
\[ l = 6 \text{ ft} \]
\[ h = 17 \text{ in.} \]
\[ f'_c = 5700 \text{ psi} \]
\[ f_y = 49.7 \text{ ksi} \]
\[ f'_y = 49.7 \text{ ksi} \]
\[ p = 1 \text{ per cent} \]
\[ p' = 1 \text{ per cent} \]
Cracking Load

Vertical 66.0 kips
Axial 11.0 kips

Ultimate Load

Vertical 136.4 kips
Axial 22.8 kips

FIG. 33 FRAME F-10
Failure in shear - compression

DATA

$c = 1/3$
$l = 6 \text{ ft}$
$h = 27 \text{ in.}$
$f'_c = 4090 \text{ psi}$
$f_y = 47.8 \text{ ksi}$
$f'_y = 48.4 \text{ ksi}$
$p = 2.33 \text{ per cent}$
$p' = 1 \text{ per cent}$
Cracking Load

<table>
<thead>
<tr>
<th>Type</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical</td>
<td>57.2 kips</td>
</tr>
<tr>
<td>Axial</td>
<td>9.5 kips</td>
</tr>
</tbody>
</table>

Ultimate Load

<table>
<thead>
<tr>
<th>Type</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical</td>
<td>70.4 kips</td>
</tr>
<tr>
<td>Axial</td>
<td>11.7 kips</td>
</tr>
</tbody>
</table>

DATA

\[
\begin{align*}
  c &= 1/3 \\
  l &= 10 \text{ ft} \\
  h &= 43 \text{ in.} \\
  f'_c &= 5000 \text{ psi} \\
  f_y &= 45.8 \text{ksi} \\
  f'_y &= 47.2 \text{ksi} \\
  p &= 2.33 \text{ per cent} \\
  p' &= 1 \text{ per cent}
\end{align*}
\]

FIG. 34 FRAME F-11
Failure in shear - compression
FIG. 35 LOAD-DEFLECTION CURVES FOR FRAMES WITH WEB REINFORCEMENT