DIAGONAL TENSION REINFORCEMENT FOR

4" SECTIONS FOR FLAT PLATE CONSTRUCTION

Thesis presented for the degree of

Master of Engineering

John E. Ellis

August
[illegible]

Civil Engineering
McGill University

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TABLE OF CONTENTS

INTRODUCTION

Abstract

The methods of overcoming the problem of high diagonal tension stresses in flat plate construction are: (1) to change the building dimensions or the concrete strength or both, (2) to use grillages, (3) to use a reinforcement system made from ordinary reinforcing bars. An increase in the concrete strength is the most economical method but does not permit the diagonal tension stress to be increased by more than 33 percent. The second method is about eight times as costly as the increase in concrete strength and about four times as costly as the third method. The second method has been certified by building codes. The third method is the subject of this thesis.

It was decided to first experiment with the reinforcement in 6" beams, the depth of 6" was considered as critical. Twelve beams were tested in which three types of reinforcement were tried. All the beams failed in bending moment and therefore the reinforcement was not tested to ultimate strength. All three types of reinforcement were effective in taking an ultimate diagonal tension stress of 0.2 f_c'.

The most suitable type of reinforcement was tested in slab specimens. The slabs were 44" square, 6" deep, with a concentric 10" column stub, and were supported along the edges. Four specimens were cast, one without diagonal tension reinforcement and three with reinforcement. The specimen without the reinforcement failed in diagonal tension and the others failed in bending moment. No ultimate strengths of the reinforcement were obtained because of the initial failure in bending moment. The diagonal tension reinforcement was effective in taking an ultimate diagonal tension

stress of 0.12 f'_c.

The slabs failed in bending moment at loads equal to two thirds of the calculated failure loads. This discrepancy between computed and actual failure loads indicates the danger in the design of flat plate buildings of assuming that the full width of the column strip is effective in resisting bending moment.

Introductory historical statement

In 1947, at McGill University, Mr. B. Gersovitz attempted to solve the problem of diagonal tension reinforcement for 6" sections for flat plate construction. He employed three types of diagonal tension reinforcement (Fig. A, B & C). His test specimens were 5' - 8" square slabs, 6" deep, with concentric 14" column stubs (Fig. D). These slabs were supported along the edges. Four specimens were cast, one without diagonal tension reinforcement and three with diagonal tension reinforcement, one slab for each type. All three types of reinforcement were ineffective.

In the autumn of 1947, Mr. B. Eskenazi attempted to ascertain if a section seven inches thick could be reinforced against diagonal tension by using individual stirrups hooked around the main steel. Four beams were cast, two without stirrups and two with stirrups. All beams failed at approximately the same load. The results indicated that further investigation was needed on diagonal tension reinforcement in thin sections.

In 1948 the Portland Cement Association suggested a reinforcement system for solving the problem of diagonal tension in 6" sections for flat plate construction. (Fig. E) (5). This reinforcement system had never been tested for strength.

Acknowledgement

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Notes to the reader

All illustrations are grouped at the end of the thesis. Reference to the photographs is indicated by (Fig. X), to the drawings by (Drwg. X), and to the bibliography by (A - B). In the reference to the bibliography, A indicates the book or article and B gives the relevant page number. It should be noted that the Appendix has a separate bibliography (see Table of Contents).

To help the reader to recognize the divisions and sub-divisions of the longer sections of this thesis, the divisions have headings in underlined capital letters and the sub-divisions have headings in underlined lowercase letters. A study of the Table of Contents will reveal the organization of this thesis.

The concrete which was used throughout the experiments was designed to be workable and to fail at 3,000 p.s.i. The appendix records the design of the mix.

FLAT PLATE CONSTRUCTION

GENERAL

Flat plate construction is similar to flat slab construction except that the column capitals and drop panels are omitted.

Flat slab construction has not been extensively used in buildings for human occupancy as the column capitals and drop panels are difficult to treat architecturally. Flat plate construction does not have column capitals or drop panels and possesses the following advantages over other types of reinforced concrete construction:

(1) reduction in forming costs
(2) reduction in cost of air conditioning installation
(3) reduction in storey height
(4) reduction in plastering and decorating costs
(5) improved lighting and acoustic conditions.

From the Clinton Hill Housing Development in Brooklyn, cost comparisons were made of the flat plate construction and the slab and beam construction. Contractor's costs of the reinforced concrete frame and floors showed that the flat plate construction was ten percent more economical than the slab and beam type of construction (10-3).

Two methods can be used for the design of flat plate structures: (9-41)

(1) Method of continuity considering the structure to be divided into a number of bents.
(2) Method of moment coefficients based on the results of tests and experience.

The first method must be used when it is impossible to meet the required uniformity of panels (9-43).

DIAGONAL TENSION IN FLAT PLATE CONSTRUCTION

In flat plate construction, a section which is of sufficient depth to resist the bending moment is not always deep enough to take the diagonal tension. The critical depth to be reinforced against diagonal tension was taken as 6". This depth is commonly used for buildings of human occupancy and it was felt that if a 6" slab could be reinforced against diagonal tension then deeper sections could be reinforced likewise.

Four methods have been advanced to solve the problem of excessive diagonal tension stresses:

(1) Increased building dimensions and concrete strength

(2) Wheeler Smooth Ceiling System

(3) Gersovitz diagonal tension reinforcement

(4) P.C.A. diagonal tension reinforcement

Increased building dimensions and concrete strength

Diagonal tension around a flat plate column is computed for a vertical section t - 1 1/2" away from the face of the column, and the unit diagonal tension stress is given by $v = V/bjd$, in which V = external shear, b = width of section and jd = moment arm between compression and tension forces in a beam. A satisfactory design can be accomplished by varying the factors that are involved:

(a) Changing the concrete mix to increase the permissible value of v.

(b) Increase the slab thickness

(c) Increase the column size.

Mr. Gersovitz has made a design based on point b and the result was a heavy and uneconomical structure. From a design using point c, Mr. Gersovitz concluded that this method was fairly economical but that the columns were excessively large. Where floor space is valuable this solution would be discarded. Point a can be used for increasing the permissible diagonal tension stress up to 33 percent, i.e. considering the maximum compressive strength of concrete as 4000 p.s.i. Assuming a 12' x 12" x 12" column and a panel of 16' x 16' x 6" = 5.5 cu.yd. and taking the cost of 4000 lb. concrete as $10.00/cu.yd. and the cost of 3000 lb. concrete as $9.20/cu.yd., the extra cost per column of increasing the diagonal tension stress by 33% is $4.50.

Wheeler Smooth Ceiling System

In this construction, patented by Mr. W.H. Wheeler, E.M., structural steel frames are buried in the slab over the columns and take the place of the drop panel and column capital of the flat slab construction (Fig. F & G).

This grillage serves the same function as the typical concrete column capital on interior columns. It decreases the shear and reduces the critical bending moment by shortening the net span. The grillage is designed to support the entire load of the slab and to resist the moment and shear.

In the early stages of the development of the grillages, comparative tests were made by the Building Department of the City of Minneapolis. Three types of slab specimens were tested and three slabs of each type were used. The first specimens had regulation concrete capitals, the second type had grillages and the third had neither capitals nor grillages. The loads carried at failure by the first two types were as identical as one could expect. In both types the failure was in a circle around the perimeter of the caps and grillages. The third type failed by the punching through of the column and at much lower loads than the slabs of the first two types.

A distinct advantage of the Wheeler Smooth Ceiling System is that conduits can be passed through the slab adjacent to the column.

The Wheeler system has been used for apartments, warehouses, hospitals and footings. Approximately 200 buildings have been constructed using this method of flat plate construction.

The cost of one grillage for a 6" slab and a live load of 40 p.s.f. is from $30.00 to $40.00.

Gersovitz diagonal tension reinforcement (4)

In 1947, at McGill University, Mr. Gersovitz attempted to reinforce slab specimens against diagonal tension.

Each test specimen was 5' - 8" square, 6" thick and had a concentric column stub (Fig. D). One specimen was used as a standard and had no diagonal tension reinforcement. Three types of diagonal tension reinforcement were used, one type per specimen.

Type 1: A continuous U-shaped stirrup placed in concentric circles about the column. This was simply an extension of the ordinary stirrups used in beams (Fig. A).

Type 2: A continuous helix placed in concentric circles about the column forming a series of toroids (Fig. B).

Type 3: A continuous N-shaped stirrup placed radially.

All the slabs failed in diagonal tension at about the same ultimate load. In all cases the plane of failure passed through some of the diagonal tension reinforcement.

P.C.A. diagonal tension reinforcement (5)

In 1948, the Portland Cement Association suggested a method of solving the problem of diagonal tension in flat plate construction (Fig. E).

The stirrups were inclined at 45 degrees and were designed according to the normal procedure for web reinforcement in compliance with the A.C.I. and Joint Committee Codes. The stirrups were welded at either end to tie rods which complied with the requirement in the A.C.I. Building Code that states that welding to longitudinal reinforcement is an approved method of providing anchorage. Further improvement in embedment in the tensile zone, where anchorage is needed most, was provided by the stirrup being continuous there.

The approximate cost of the P.C.A. reinforcement for one column is $10.00 or about one quarter of the cost of the Wheeler grillage (see below under "Cost of Manufacture of B, D and E Types").

A modified form of the P.C.A. reinforcement was tested in the course of the work for this thesis.

Conclusions

When increases in diagonal tension strength of less than 33% are all that are required, consideration should be given to increasing the strength of the concrete. This solution of the problem of diagonal tension in flat plate construction would cost about $5.00 per column.

When the diagonal tension stresses are severe, the Wheeler grillages can be used at a cost per column of from $30.00 to $40.00.

After a study of this thesis, an engineer could decide whether or not the modified form of the P.C.A. reinforcement could be used. The estimated cost of this method of overcoming the problem of diagonal tension in flat plate construction is $10.00 per column.

FUNDAMENTALS OF DIAGONAL TENSION

Introduction

The fundamental facts and ideas of diagonal tension that are given in this section were obtained from a study of the bibliography.

Concrete in tension

Concrete is strong in compression and has an ultimate shear strength equal to about one half its strength in compression. The ultimate tension strength of concrete is not more than one tenth of its ultimate compression strength. Thus concrete is weak in tension.

Principal stresses in a reinforced concrete beam

At any point in a beam the maximum tensile stress is $1/2\ f + \sqrt{1/4\ f^2 + v^2}$, in which f = the horizontal fibre stress and v = the shear stress.

In a reinforced concrete beam f is not distributed as in an homogeneous beam because the tensile fibre stresses are concentrated in the steel. When the tensile fibre stresses are sufficiently high to stress the steel to its yield point, the concrete in the tension region is cracked and all the tensile fibre stresses, except a negligible amount, are concentrated in the steel. Neglecting the tensile fibre stresses carried by the concrete, the maximum tensile stress in the concrete equals the shear stress. At the neutral axis, where tensile fibre stresses do not occur, this relationship holds exactly. The maximum tensile stresses theoretically occur along a plane at 45 degrees to the neutral axis of the beam; owing to the angle of their occurrence, these stresses are known as diagonal tension stresses. The nominal formula that relates the external shear and the diagonal tension stress is $v = V/bjd$, in which v = the nominal diagonal tension stress, V= external shear, b = width of section and jd = the moment arm between the tension and compression forces in the beam. This nominal formula neglects the tension stresses taken by the concrete and is based upon the laws of statics. All standard texts show the derivation of this nominal formula.

Diagonal tension taken by the concrete

In a simple beam that is designed to fail in diagonal tension and that does not have web reinforcement, the failure usually follows closely upon the formation of the first diagonal tension crack.

Experiments on beams with web reinforcement have shown that in beams with diagonal tension cracks, the nominal diagonal tension force was greater than the actual force carried by the web reinforcement. This indicated that concrete takes a portion of the diagonal tension stress even at high loads and in a cracked condition. This indication has been corroborated by beam tests in which the web reinforcement was kept constant but the width was different for specimens that were otherwise identical. All specimens failed in diagonal tension; the wider specimens failed at higher loads which proved that concrete takes diagonal tension at high loads.

The ultimate diagonal tension strength, as calculated by the nominal formula, is between five and ten percent of the compressive strength of concrete. These values hold for footings and slabs in which the nominal diagonal tension stress is calculated for a section removed from the face of the column by a distance equal to the effective depth of the footing or slab.

Web reinforcement

Web reinforcement prevents diagonal tension cracks from opening up to a dangerous extent; it increases the ultimate diagonal tension strength of the structure.

An ultimate diagonal tension stress, as calculated by the nominal formula, of about 20 percent of the compressive strength of concrete can be obtained by using vertical stirrups. For stirrups placed at 45 degrees to the neutral axis, an ultimate diagonal tension stress of about 30 percent of the compressive strength of concrete can be attained. If these diagonal stirrups are welded to the longitudinal steel, the ultimate diagonal tension strength is increased to about 35 percent of the compressive strength of concrete.

For the design of web reinforcement the concrete is considered to take an approximate ultimate stress of 0.005 f_s', in which f_s' is the unit stress in the web reinforcement.

Inclined stirrups and bent up bars take diagonal tension stresses even when the beam has no diagonal tension cracks. The stress in these two types of web reinforcement increases greatly upon the formation of the first diagonal tension crack. Vertical stirrups normally receive stress only upon the formation of diagonal tension cracks.

Test results of strain gages on web reinforcement show that the readings are influenced greatly by where the cracks cross the web reinforcement. The cracks show the critical regions of diagonal tension.

<u>Influence of bond</u>

The nominal bond stress is given by the formula $u = V/Ojd$, in which V = total external shear, O = sum of the perimeters of the reinforcing bars, and jd = moment arm of the compression and tension forces in the beam. The nominal diagonal tension stress is $v = V/bjd$, in which V and jd are as above and b = width of the section. From the above two formulae $vb = Ou$. As b and O are fixed for a given beam with a given number and type of reinforcing bars, any change in the bond stress causes a corresponding change in the diagonal tension.

In beam tests, the initial slip of the horizontal bars, the appearance of diagonal tension cracks, and the development of high stresses in the stirrups, have been observed to occur simultaneously. Bond tests on beams have shown that the maximum bond stress that can be maintained occurs at an end slip of 0.001 inches. The reason for increasing the permissible diagonal tension stresses when hooks are used is because it is believed that many supposed diagonal tension failures have been bond failures in disguise. Ultimate values for bond stresses of plàin round bars and hooks are given in Prof. Mylrea's article on Bond and Anchorage (12). His values for flexural bond stresses apply to simple beams where the end slip of the horizontal bars is less than 0.001 inches.

The specifications require the allowable bond stress to be reduced 25 percent in all tension regions having moment reinforcement in more than one direction. Bond in tension concrete is weaker than bond in compression concrete.

Deformed bars with good deformation characteristics will develop about double the bond stress of plain round bars .

Influence of horizontal steel

The elongation of the horizontal steel, when the beam is loaded by an amount equal to the design load, may be sufficient to produce cracks in the concrete. These cracks interrupt the bond between the concrete and the steel and cause the bond stresses near the middle of a simply supported beam to decrease and necessarily cause the bond stresses near the end portions of the beam to increase. Thus the elongation of the steel changes the distribution of the bond stresses. As the bond stresses and the diagonal tension stresses are related, the elongation of the steel causes an increase in the diagonal tension stress in the end regions of beams.

In footings without web reinforcement, tests have shown that when initial failure was in tension, the secondary failure was in diagonal tension and occurred at abnormally low diagonal tension stresses. The yielding of the steel resulted in cracks which reduced the area of the section resisting diagonal tension. Cracking occurs when steel strains are excessively large, this cracking reduces the area of the section resisting diagonal tension and precipitates failure by diagonal tension.

Footing tests have shown that the maximum diagonal tension stresses for footings increase with the amount of horizontal reinforcement. An explanation is that part of the resistance to diagonal tension is provided by the dowel action of the bars which cross the section of failure.

Deflection

Stirrups decrease the deflection of beams by about four percent for small loads. For larger loads the reduction is greater. The shear stresses and the opening of diagonal tension cracks affect the deflection.

PLAN OF EXPERIMENTS

The problem of reinforcing flat plate construction, having 6" slabs, against diagonal tension was resolved into first reinforcing 6" beams against diagonal tension. The following reasons account for the decision to first attempt to reinforce 6" beams against diagonal tension rather than to first attempt to reinforce 6" flat slab specimens against diagonal tension.

(1) The phenomenon of diagonal tension failure appeared to be the same for simple beams, for continuous beams and for flat plate construction (4-55, 16-123). It was felt that if simple beams could be successfully reinforced, then flat plate construction could be reinforced likewise.

(2) Beams were easier to construct and test than slab specimens.

(3) Mr. Eskanazi's work on reinforcing 7" beams against diagonal tension indicated that the problem of reinforcing shallow sections against diagonal tension had not been solved (4-65).

(4) In beam specimens the diagonal tension cracks are visible along the two sides of the beam. This advantage to the study of the problem would not hold for slab specimens.

In the beam tests the diagonal tension reinforcement was designed so that it would be adaptable to flat plate construction.

The experiments, which are recorded in the next sections of the thesis, took the following general pattern:

Beams A & B - preliminary tests to ascertain the value of a modified form of R/C No.22 reinforcement.

Beams C, D, E & F - tests to experiment with two other types of diagonal tension reinforcement and to determine the diagonal tension stresses in the modified R/C No.22 reinforcement.

Slabs G, H, I & J - to experiment with slab specimens, using the most suitable type of diagonal tension reinforcement. These specimens resembled the portion of a flat plate building around the top of the column.

BEAMS A & B

PURPOSE

These beams were made and tested in order to ascertain the value of a modified form of the R/C 22 diagonal tension reinforcement.

DESIGN (Drwg.1)

Number of specimens

Four beams were made which were identical except that beams A-1 and A-2 did not have diagonal tension reinforcement and beams B-1 and B-2 did have diagonal tension reinforcement.

Width (Fig.1&2)

To simulate flat plate construction in which diagonal tension stresses occur over widths of six feet or more, the specimens were made as wide as possible. The maximum width, limited by the testing machine, was 18 inches.

Depth and covering for reinforcement

The specimens had a total depth of six inches and a covering for the reinforcement of 3/4 of an inch.

Length

Three factors were considered in determining the length of the specimens. Firstly, third point loading was desired in order to separate the cracks caused by tensile fibre stresses from the cracks caused by diagonal tension stresses. Secondly, at least two rows of stirrups were needed in order to make the test truly representative. The stirrup spacing was t-1.5" = 4.5 inches, therefore the distance from a loading point to an adjacent support was nine inches. As will be realized later, a distance greater than nine inches would mean a prohibitively high bending moment.

Thirdly, the panel had to be carried past the supports to fulfil the requirements for the anchorage of the tensile steel. An overhang of 9 1/2" at each end was required for anchorage.

Overall dimensions (Drwg. 1)

Width = 18"

Length = 46"

Depth = 6"

Effect of the dead weight

The effect of the dead weight was determined to ascertain if it should be considered in the design of the beams. Considering a width of 12", the dead load = 75 lbs./ft. = 6 lbs./inch. The moment caused by the dead load $= wl^2/8 = 6x27^2/8 = 550$ in.lbs. which was 0.25% of the design moment and therefore negligible. Assuming jd = 3.5", the diagonal tension stress due to dead weight = 27x75/2x12x12x3.5 = 2 p.s.i., which was 0.6% of the design diagonal tension stress and therefore negligible.

Tensile fibre stresses taken by the concrete

The ultimate strength of concrete in tension was taken as 0.1 f'_c or 300 p.s.i. A width of 12 inches was considered and the steel area was taken as 4.99 sq.inches (see below in section on design of tension steel). The covering of the steel was 3/4 inches, the value of n was taken as 10, the thickness of the tensile steel was 1 1/4" and the effective depth was 4.625 inches (Drwg. 2). The neutral axis, $\bar{x}$, was located as follows : $\bar{x} = \sum ax/\sum a$ = (12x6x3 + 45x4.625)/(12x6+45) = 3.62". The total moment of inertia, I, equals the moment of inertia of the concrete plus the moment of inertia of the steel. $I_c = bd^3/12 + Ad^2 = 12x6^3/12 + 12x6x0.62^2 = 243.6$. $I_s = bd^3/12 + Ad^2$, in which b = 45/1.25 = 36", therefore $I_s = 36x1.25^3/12 + 45 \times 1.005^2 = 51.37$. Thus $I = I_c + I_s = 295.0$. The moment at which the concrete fails in tension = If/c = 295x300/2.38 = 37,000 in.lbs. which is 15 percent of the design moment. Therefore the effect of the concrete in taking tensile fibre stresses is negligible.

Tension steel

The tension steel was designed to carry a load which would cause a diagonal tension stress of at least 0.06 x f'_c x 3 = $0.18f'_c$. The maximum diagonal tension stress, which Mr. Gersovitz computed, for flat plate construction was slightly less than $0.06f'_c$. The safety factor was taken to be three.

Balanced design

In designing the steel for a balanced section, the following assumptions were made: fs = 40,000 p.s.i., f_c = 3,000 p.s.i., n = 10, size of bars = 3/4", covering on bars = 3/4". A width of section of 12" was taken for convenience in computations; the effective depth was 4.75". The distance to the neutral axis was x and 3000/x = 7000/4.75, whence x = 2.03". The compressive force = 1/2x3000x2.03 = 30,400 lbs., and jd = 4.75-2.03/3 = 4.07". Thus the moment = 30,400x4.07 = 124,000 in.lbs. The shear = 124,000/9 = 13,800 lbs. and the diagonal tension stress = 13,800/12x4.07 = 283 p.s.i. which equals $0.10f'_c$. This relatively low diagonal tension stress is explained by the fact that in reinforced concrete beams, the moment varies with d^2 but the diagonal tension varies with d. Hence for small values of d the resisting moment governs in the case of simple beams. This relationship between the resisting moment and the diagonal tension does not hold for continuous beams and for flat plate construction.

Balanced design did not provide for a load which would cause a diagonal tension stress of $0.18f'_c$. To increase the resistance of the section to bending moment in order to provide a diagonal tension stress of at least $0.18f'_c$, two solutions presented themselves. The first was to use very heavy tensile reinforcing and the second was to use moderately heavy tensile reinforcing plus compression steel. In beams A and B it was decided to use the very heavy tensile reinforcement for the following reasons:

(1) The elongation of the tensile steel was to be kept a minimum so that the elongation would not crack the concrete and thereby weaken the section resisting diagonal tension.

(2) As steel does not always occur in the compression zone over the column in flat plate construction, it was not desirable to use compression steel in these specimens.

(3) As the compression steel would be close (4") to the tension steel it might help reinforce the section against diagonal tension. Beams C confirmed this supposition.

(4) A high steel percentage would mean a low steel stress and therefore minimum requirements for bond, Hooks would not be required. This was desirable as hooks, in helping to restrain the section against diagonal tension, would complicate the problem.

Design of section by straight line theory

The very heavy tensile reinforcement was 1 1/4" square bars at 3 1/4" centers giving 4.99 square inches of steel per 12" width of section. A 12" width was considered in the computations. The effective depth = 6-3/4-5/8 = 4.625". Assuming n = 10, the distance to the neutral axis, y, was found as follows:

$$12y^2/2 = 10x4.99\ (4.625-y).$$

$$6y^2 = 231-50y$$

$$6y^2+50y = 231$$

$$y^2+8.33y = 38.5$$

$$y^2+8.33y+(8.33/2)^2 = 38.5+(8.33/2)^2$$

$$y+4.16 = 7.46$$

$$y = 3.3$$

The compressive force taken by the concrete, assuming f_c = 3,000 was 1/2x 3,000x12x3.3 = 59,500 lbs. The resisting moment of the concrete was 59,500(4.625-3.3/3) = 210,000 in.lbs. The corresponding shear is 210,000/9 = 23,300 lbs. The diagonal tension stress = 23,300/12x(4.625-3.3/3)= 550 p.s.i. = 0.18 f'_c which is satisfactory. The steel stress for this moment = 3,000x10x(4.625-3.3)/3.3 = 12,000 p.s.i.

With such heavy reinforcement it was questioned whether ordinary beam action would occur. As the steel alone would only support a load equal to 13 percent of the above design load, it was concluded that normal beam action would take place.

Design of steel by exact formulae (1-75)

The following table summarizes the designs for tensile steel of 1 1/4" square bars at 3 3/4". The covering of the steel is 3/4". The values apply to the total beam width of 18".

Formula	Moment in.kips	Steel Stress p.s.i.	Shear kips	Diag. p.s.i.	Tension &f'_c	jd
Str.Line	310	12,000	34.5	550	0.18	3.52
Rectangular	384	15,000	42.8	705	0.23	3.38
Parabolic	320	11,600	36.0	540	0.18	3.72
Cubic	400	17,000	44.2	780	0.26	3.14
Quintic	380	13,800	42.0	640	0.21	3.63
Average	360	14,000	40.0	600	0.2	3.48

As 1 1/4" square bars could not be obtained, 1 1/4" round bars were used. The actual steel area per 12" width was 4.91 square inches instead of the 4.99 square inches used in design. This discrepancy of 1.7 percent did not warrant a re-design.

Bond for tensile steel

The computed maximum steel stress was 14,000 p.s.i. From

Prof. Mylrea's article on bond and anchorage (12-533) an l/d value of 8 for third point loading should develop a stress of 14,000 p.s.i., in which l is the length of the bar measured from the reaction and d is the diameter of the bar. For 1 1/4" bars the anchorage distance for bond was 8x1.25 = 10". To provide the required distance and to also assure a small safety factor in bond, the bars were extended 9" past the support.

Diagonal tension reinforcement

Design of stirrups

The diagonal tension stress was taken as $0.2f'_c$ = 600 p.s.i. The ultimate tensile stress in the web reinforcement was considered to be 40,000 p.s.i. The concrete was assumed to carry a diagonal tension stress of 0.005 f'_s = 0.005x40,000 = 200 p.s.i., this unit stress corresponds to the strength of concrete in diagonal tension of 0.05 f'_c to 0.1 f'_c = 150 p.s.i. to 300 p.s.i. Therefore the unit diagonal tension stress to be taken by the stirrups was 400 p.s.i. The stirrups were at 45 degrees to the horizontal and were spaced at a distance of t-1.5 = 4.5". The web reinforcement was designed to take a diagonal tension force for a 12" width of 400x12x4.5xsin45 = 15,300 lbs. For an ultimate tensile stress of 40,000 p.s.i., the cross sectional area of the stirrups for a 12" width = 15,300/40,000 = 0.38 sq.in. The stirrup was of a V shape as suggested in R/C 22. It was designed to have a concrete covering of 3/4" on the tension side and to have no covering on the compression side.(Drwg.3). From the drawing it is seen that the web reinforcement had four legs per 12" width. Therefore the cross sectional area of each leg was 0.38x8.0/ 7.5x4 = 0.1 sq.in. A 3/8" stirrup provides 0.115 sq.in. and was suitable.

Anchorage for stirrups

It was assumed that the length of the anchorage in the compression region was 0.2 t = 2". According to Prof. Mylrea, the stress that can be developed in the stirrup due to this anchorage is about 8,000 p.s.i. A stress of 40,000 p.s.i. was required. Therefore anchorage by bearing had to be used in the compression zone. In the tension zone the anchorage was also obtained by bearing because the stress of 40,000 p.s.i. had to be developed at the level of the tensile steel.

In attempting to ascertain the strength of concrete in bearing, three sources of information were used:

(1) Tests made by Prof. Richart (15-34).

(2) Prof. Mylrea's conclusions about the crushing strength of the concrete surrounding the hooks of reinforcing bars(12-549).

(3) Relevant tests carried out at McGill University by the writer.

(1) In tests of ordinary vertical stirrups, Prof. Richart purposely left a small space between the stirrup loop and the longitudinal steel. He examined this space to see if the concrete, which filled the space, was crushed when the stirrup was highly stressed. No crushing was noted, the inference being that very high bearing pressures (5,000 p.s.i.) may be sustained by a small area of concrete that is well restrained by the surrounding mass of the concrete.

(2) Prof. Mylrea states that for normal hooks on reinforcing bars, a tangency stress of 20,000 p.s.i. is sufficient to cause a crushing of the concrete. Consider a 1" round bar and assume the diameter of the hook to be three inches. Assume that the hoop formula holds: $p = qr$ where p = hoop tension, q = bearing stress and $r = 1.5"$. Whence $q = 20{,}000 \times 3.14/1.5 \times 4 = 10{,}500$ p.s.i. This would indicate that a bearing stress of 10,500 p.s.i. can be attained.

(3) Six specimens, which consisted of welded cross bars imbedded in concrete

blocks, were made and tested to help solve this problem. (Drwg. 4 and Fig. 3 and 4). The 1/4" specimens failed at 700 lbs., 2800 lbs. and 2700 lbs. The 1/2" specimens failed at 4,520 lbs., 3,000 lbs., and 2,800 lbs. The one half inch specimens were qualitatively superior; the average bearing stress of the concrete for these specimens was 1800 p.s.i. All specimens failed by splitting the concrete. And the chief value of these tests was to show that the important factor is the degree of restraint provided by the adjacent concrete.

From a consideration of the above three sources of information on the bearing strength of concrete, it was decided to take the bearing strength of concrete as 3,000 p.s.i.

Anchorage by bond was neglected and the bearing stress on the tie rod was assumed to be uniformly distributed along its length. The diagonal tension force for a 12" width = 15,300 lbs; therefore the size of the tie rod = 15,300/3,000x12 = 1/2".

To accomodate the grid of tensile steel which occurs in flat plate construction, the tension tie rods were placed with a clearance of 2 1/4" so that two layers of 3/4" bars could be placed and a 3/4" covering could be obtained. The compression tie rods were placed with a clearance of 3/4" (Drwg. 1). They should have been placed with the same clearance as the tensile tie rods, because a reinforcing grid occurs in the compression zone in the region of the shearhead in flat plate construction.

Tests were carried out to determine whether or not the weld joining the 1/2" tie rod and the 3/8" stirrup was sufficiently strong to develop the total design load in the stirrup of 4,100 lbs. (i.e. 15,300x 8.1/4x7.5 = 4,100). The six specimens were made of bars which were samples taken from the material used for web reinforcement in the B beams. This material was Intermediate Grade Steel, New Billet. (Drwg. 5). Arc welding was used for all specimens; it was done by the Dominion Welding and Engineering Co. of Montreal. The four types of welds, using the designation of the American Welding Society, were E - 4520, E - 6010, E - 6011, and E - 6012. All electrodes were 5/32" in diameter.

All six specimens gradually took the shape shown in Drwg.6 between loads of 2,500 and 3,000 lbs. Failure occurred at section A-A when the bending was more acute than shown in Drwg.6 (Fig.5). The weld did not fail in any specimen. In the following table the data of the tests are given (Fig.6).

Electrode	Failure load
E - 4520	3230 lbs.
E - 6010	4640
do	4460
E - 6011	3000
do	3660
E - 6012	3120

These tests indicated that the welds provide sufficient strength. The critical factor was not the strength of the weld but the effect on the 3/8" bar of the heat generated in welding. The E - 6010 electrode was satisfactory and was used in welding the web reinforcement.

Spacing of web reinforcement

The spacing of the web reinforcement was t-1.5" = 4.5". To comply with the C.E.S.A.'s specifications (13), the spacing adjacent to the support was taken as one half the normal spacing (Drwg.1).

BEAMS A

Gages

Deflection and bond gages were used; all gages were Ames dial gages which read in 1/1000 inches. The single deflection gage was placed at the center of the bottom of the panel. For each panel two bond gages were mounted which indicated the end slip of the tension steel. Unfortunately they were both placed at the same end, a better arrangement would have been to have one at either end. Drawing 7 illustrates the bond gage which measured the end slip (Fig.1).

Loading (Fig.1)

The A panels were tested after an interval of 7 days.

Test data of beam A-1

The test cylinders failed at 92,700 and 80,300 lbs. giving an average compressive strength of 3,060 p.s.i. The following table presents the test data; both of the bond gages were located at the end remote from the diagonal tension failure.

Load kips	Deflection 1/1000"	Bond Gages 1/1000"	
		A	B
4	99	99.2	0.0
8	109	do	do
12	117	do	do
16	124	do	do
20	130	do	do
24	136	do	do
28	140	do	do
32	144	do	do
36	149	do	do
38	151	do	do
40	154	do	do
44	158	do	do
48	162	do	do
52	166	do	do
56	170	do	do
60	176	do	do
64	183	do	do
68	188	do	do
72	197	do	do
74	230	do	do

Test phenomena of beam A-1

No visible cracks formed prior to the failure load of 74 kips. Drwg.8 shows the diagonal tension cracks, as they appeared on both sides of the beam. Upon breaking up the beam it was seen that:(1) the plane of failure was of constant shape across the width of the panel (Fig.7), (2) at the top near the load point, the crack was almost vertical, (3) the diagonal tension failure occurred at approximately 30 degrees to the horizontal, (4) many of the individual stone particles were broken and the plane of failure of these stone particles was smooth.

The nominal diagonal tension stress at failure = V/bjd = 74,000/2x18x3.48 = 592 p.s.i.

<u>Test data of beam A-2</u>

This specimen was tested after a seven day interval. The test cylinders failed at 72,700 lbs. and 77,800 lbs. giving an average compressive stress of 2,700 p.s.i. The bond gages, whose readings are recorded in the following table, were located at the end of the beam in which diagonal tension failure occurred.

Load kips	Deflection 1/1000"	Bond Gages 1/1000"	
		A	B
4	27	42.0	65.9
8	36	do	do
12	43	do	do
16	48	do	do
20	52	do	do
24	55	do	do
28	60	do	do
32	63	do	do
36	67	do	do
40	71	do	do
44	74	do	do
48	78	do	do
52	82	do	do
56	86	do	do
60	92	do	do
64	97	do	do
68	103	do	do
72	110	do	do
76	117	do	do
80	126	42.0	65.5
84	210	0.0	2.0

Test phenomena of beam A-2

No visible cracks formed prior to the failure load of 84 kips. The diagonal tension cracks were similar to those of beam A-1 (Drwg.8).

The diagonal tension cracks and the bond failure, denoted by the end slip recorded by the bond gages, occurred simultaneously.

Upon breaking up the specimen, the same features were noticed as for beam A-1.

The nominal diagonal tension stress at failure = 84,000/2x 18x3.48 = 670 p.s.i.

Discussion of results of beams A-1 and A-2.

Both specimens failed suddenly in diagonal tension at an average diagonal tension stress of 631 p.s.i.

In beam A-1 the bond gages, which were remote from the failure end, showed no bond failure. In beam A-2 the bond gages, which were at the failure end, showed that end slip occurred simultaneously with the diagonal tension failure. This demonstrates the close relationship between diagonal tension and bond.

BEAMS B

Gages

SR-4 gages were attached to the horizontal steel to determine the change in tensile stress from the support to the point of maximum bending moment underneath the load point. Theoretically the steel stress varies directly with the moment and the shear equals the change in bending moment (V = dM/dX). Thus if the change in steel stress is known, the shear can be found. At the neutral axis the diagonal tension stress equals the shear stress. With these relationships the diagonal tension stress can be approximately determined from a knowledge of the stresses in the horizontal steel. The tensile fibre stresses in the concrete prevent this method of determining the diagonal tension stresses from being foolproof.

Because of the excessive thickness of the tensile steel, half the strain gages were placed on the top of the bars and the other half at mid depth.

In order to find the position of the neutral axis, three SR-4 gages were placed on one side of each beam (Drwg. 10).

Bond gages to measure end slip were placed at one end of beam B-1 and at both ends of beam B-2. These gages were the same as those used in beams A.

A deflection gage, placed in the center of the bottom, was used for each specimen.

Test data of beam B-1

The test cylinders, which were broken after a seven day interval, failed at 83,300 lbs. and 62,100 lbs., giving an average compressive stress of 2,600 p.s.i. The second cylinder was badly honeycombed which indicated insufficient rodding.

The positions of the internal SR-4 gages are shown in drawing 9 and the position of the external SR-4 gages are shown in drawing 10.

The following table contains the gage readings for the test of beam B-1 (Drwg. 9).

Load kips	Defl. 1/1000"	Bond Gages 1/1000"		External Gages micro-in./in.			Internal Gages micro-in./in.			
		A	B	3"	1/2"	2"	8	6	4	3
4	69	10.5	60.6	6030	7825	9125				
8	80	do	do	6010	7815	9125				
12	90	do	do	6015	7800	9120	4950	6460	5335	5560
16	96	do	do	6020	7765	9105	4970	6460	5340	5565
20	103	do	do	6035	7740	9100	4985	6460	5325	5570
24	107	do	do	6040	7710	9090	5010	6465	5325	5570
28	114	do	do	6065	7685	9085	5040	6475	5350	5575
32	119	do	do	6085	7660	9085	5065	6473	5350	5580
36	124	do	do	6095	7630	9070	5090	6480	5370	5580
40	128	do	do	6215	7600	9060	5100	6485	5380	5585
44	134	do	do	6335	7565	9050	5140	6510	5380	5595
48	138	do	do	6490	7540	9050	5170	6530	5390	5600
52	143	do	do	6670	7510	9030	5200	6550	5380	5600
56	148	do	do	6780	7470	9025	5240	6580	5400	5615
60	153	do	do	6885	7435	9010	5270	6610	5395	5630
64	161	do	do	11350	7400	9010	5350	6650	5350	5700
68	165	do	do	9400	7360	9000	5390	6660	5360	5740
72	170	10.1	60.5	9140	7320	8940	5440	6670	5360	5790
76	175	10.0	do	11235	7280	8920	5480	6690	5360	5830
80	182	9.2	do	12000	7230	8900	5540	6700	5350	5860
84	186	do	do	8450	5870	7620	5630	6760	5330	5890
88	195	8.1	do	10070	7130	8860	5660	6770	5330	5890
92	200	7.8	do	11960	7080	8840	5710	6815	5300	5870
96	207	7.2	60.5	10880	7030	8810	5770	6865	5275	5865
100	218	6.0	do	5640	6910	8760	5860	6920	5270	5850
104	224	5.3	do	5730	6850	8680	5960	6980	5250	5860
108	238	3.3	do	6050	6770	8630	6050	7030	5220	5880
108.5	300	-3.0	60.5	6560	6840	8730	5870	6920	5170	5880

Test phenomena of beam B-1 (Drwg.11).

The failure was considered to have been caused by a combination of compression and direct shear (Drwg.11).

Upon breaking up the specimen it was seen that the failure crack, which extended across the width of the beam, did not cross the diagonal tension reinforcement.

Test data of beam B-2

The test cylinders, which were broken after an interval of 7 days, failed at 99,400 lbs. and 66,000 lbs., giving an average compressive stress of 2,900 p.s.i. The second cylinder was badly honeycombed which indicated insufficient rodding.

The position of the internal SR-4 gages and the bond gages are given by drwg.12 and the position of the external SR-4 gages are given by drwg.10.

Load kips	Defl. 1/1000"	Bond Gages 1/1000"				External Gages micro-in./in.			Internal Gages micro-in./in.			
		A	B	C	D	1/2"	2"	3"	5	7	2	1
6	11	88.1	83.0						5120	5300	6410	5240
12	25	do	do						5130	5280	6430	5245
18	34	do	do				7280	7870	130	280	450	250
24	43	do	do				7310	7880	130	285	480	250
28	52			42.2	71.2		7290	7880	140	300	500	255
34	58	do	do	42.0	72.0		7280	7880	150	330	540	260
40									180	350	580	270
46	71	do	do	42.0	72.0	8510	7250	7880	210	360	610	280
52	77	do	do	do	do	8420	7230	880	250	370	650	300
58	83	do	do	do	do	8330	7200	880	5310	380	700	340
64	89	do	do	do	do	8260	7180	880	5380	5380	6740	1390
70	96	do	do	do	do	8180	160	880	5440	380	6810	5430
76	105	88.1	83.0	do	do	8040	7090	870	5490	5360	6890	5480
82	114	do	82.5			7900	7030	840	5510	5360	6970	5510
86	125	do	81.1	42.0	72.0	7800	7000	830	510	290	7040	5540
90	132	do	80.7	do	do	7690	6860	7770	5500	270	7090	5570
94	138	do	80.0	do	do	7650	6830	7760	5500	5250	7160	5600
98	155	87.0	76.1	do	do	8290	6800	7850	5460	5120	7290	5650
102	170	85.5	68.5	do	do	8390	6820	7880				

Test Phenomena of beam B-2 (Drwg.13)

The failure was considered to have been initially in compression. The concrete on the top surface was easily spalled off by hand. The failure cracks extended to the top of the beam only upon the removal of the load (Drwg.13).

All diagonal tension cracks closed about 50 percent upon the failure of the specimen. Upon breaking up the specimen, no internal diagonal tension cracks were found. The probable reason for this was the elastic action of the web reinforcement.

Discussion of results of beams B-1 and B-2

Judging by the external diagonal tension cracks, all stirrups were stressed. Owing to the initial failure in compression, the tests did not reveal the ultimate strength of the stirrups. The average maximum diagonal tension stress was 840 p.s.i. or 0.36 f_c'.

End slip occurred only where the gages were on bars having strain gages 9" from the end. These strain gages interrupted the bond for 3" on one half the circumference of the bar.

In beam B-1, the bond over a length of 18" was sufficient to develop a steel stress of about 30,000 p.s.i. For B-2, the bond over a length of 18" developed a steel stress of 27,000 p.s.i. The stress developed by the 9" anchorage was 10,500 p.s.i. for B-1 and 12,000 p.s.i. for B-2. The average interior bond stress was 17,300/3.14x1.25x9 = 490 p.s.i. and the average anchorage bond stress was 11,200/3.14x1.25x9 = 318 p.s.i. These figures are in accord with Prof. Mylrea's values (12-533+526).

The average interior bond stress was 490 p.s.i. The average longitudinal shear stress above the bars due to this bond stress was 490x3.14x1.25x6x9/18x9 = 650 p.s.i. But the average diagonal tension stress was 840 p.s.i. The latter figure is theoretically correct for the neutral axis. The difference between the two values could be attributed to the tensile fibre stresses taken by the concrete below the neutral axis and to any inaccuracies of the SR-4 gages.

The angle of the diagonal tension cracks was, in general, around 35 degrees. Theoretically it should be 45 degrees; this discrepancy indicated the presence of other forces such as compression.

The external strain gages showed that the neutral axis was from 3.0" to 3.2" from the top of the specimens. The straight line formula was the most accurate in predicting the position of the neutral axis.

The average maximum tensile stress in the steel was 28,000 p.s.i. which gave a maximum force carried by the steel = 6x3.14x 1.25x1.25x28/4 = 208 kips. The maximum moment = 53x9 = 478 in.kips, jd = 4.625-0.4x3.2 = 3.38 and the compressive force = 478/3.38 = 142 kips. It is evident that a serious discrepancy occurred between the tension and the compression forces.

The average stress in the horizontal steel between the reaction and the load point upon the formation of diagonal tension cracks was 23,000 p.s.i.

A study of the deflection results showed that for loads up to 70 kips, the deflections for both beams A and B were approximately the same.

Problems arising from tests of beams A and B

The first problem was that of explaining the abnormally high diagonal tension stresses at which beams A-1 and A-2 failed and at which diagonal tension cracks appeared in the beams B-1 and B-2. Beams A-1 and A-2 failed in diagonal tension at an average diagonal tension stress of 631 p.s.i. Diagonal tension cracks appeared on the sides of beams B-1 and B-2 at an approximate diagonal tension stress of 550 p.s.i. The usual failure stress in diagonal tension is between 200 and 300 p.s.i.

Firstly, the high early cement was investigated to ascertain if it is stronger in tension than ordinary 28 day cement. From their data on high early cement tests for January and February 1949, the chemists of the Canada Cement Company supplied the following information in which "ratio" is the compressive value of cement prisms to the tension value of cement briquettes. The figures given in the following table represent the average of a month's tests.

Cement	Month	Ratio
High Early	January	7.79
Ordinary	do	4.78
High Early	February	8.13
Ordinary	do	4.44

The above findings indicate that high early cement is relatively weak in tension. It was concluded that the cement did not account for the abnormally high diagonal tension stresses taken by the concrete.

Secondly, the concrete itself was investigated to determine its tensile strength. Six briquettes and three controlling cylinders were poured (Drwg. 14 and Fig. 8). The six briquettes failed suddenly, without any cracks appearing before failure, at the following stresses in p.s.i.: 170, 280, 280, 240, 120, 170, (Fig. 9). The average tension stress was 210 lbs. The average compression stress of the three controlling cylinders was 2,840 p.s.i. It was concluded that the concrete did not account for the abnormally high diagonal tension stress.

Thirdly, an attempt was made to correlate the elongation of the steel, the formation of concrete cracks and the bond failure between the steel and the concrete. Three specimens were made at the same time as the tension test specimens (Drwg. 15). In two of the specimens, haircracks formed at the middle of the specimen and extended around the perimeter of the concrete at right angles to the steel at a unit stress in the steel of approximately 20,000 p.s.i. In the third specimen, no external cracks were visible. In all three specimens the bond failed entirely at a steel stress of approximately 40,000 p.s.i. (Fig. 10).

The third group of tests gave a clue to the solution of the problem. In a shallow section, such as the one under consideration, the diagonal tension stresses would be transferred to the steel as long as the bond was unbroken. Another way of expressing this idea is by stating that with the steel so close to the neutral axis, it restrains the concrete against cracking from diagonal tension as long as the bond holds. The bond held for beams A and B because (1) the horizontal steel did not elongate sufficiently to break it and (2) the diagonal tension stresses transferred by bond were not excessive.

The second problem that arose from the tests of beams B-1 & B-2 was the high observed force carried by the tensile steel. According to the SR-4 gages on the horizontal steel, the maximum force carried by the steel was 208 kips, yet the compressive force was only 142 kips. This discrepancy was explained by the gage being under the load point where the excessive deflection probably accounted for the abnormally high strains registered by the SR-4 gages.

CONCLUSIONS OF BEAMS A & B

In beams A-1 and A-2 the concrete resisted a diagonal tension stress of 0.21 f'_c. These beams, which had no diagonal tension reinforcement, failed suddenly in diagonal tension.

In beams B-1 and B-2 the web reinforcement was effective in resisting a diagonal tension stress of 0.36 f'_c. The ultimate strength of the web reinforcement was not ascertained as the beams failed in compression.

BEAMS C, D, E, and F

PURPOSE

The tests were conducted in order to experiment with two other types of web reinforcement and to determine the diagonal tension stresses taken by the stirrups of the web reinforcement used in beams B-1 and B-2 (this type of web reinforcement will be called the B-type reinforcement).

DESIGN

The number and the general description of the specimens

Two beams of each type were made. Beams C-1 and C-2 had no web reinforcement and served as standards. Beams D-1 and D-2 had web reinforcement made from expanded metal and beams E-1 and E-2 contained web reinforcement of sand-blasted steel plate. Beams F-1 and F-2 had the B-type web reinforcement on which extra small SR-4 gages were placed in order to ascertain the stress carried by the stirrups.

Width

No advantage was gained in making beams A and B as wide as 18 inches, therefore beams C, D, E and F were made 9 inches wide.

Length and loading

The length of the specimens was 46". The reasons for this length were the same as those given for the length of beams A and B. Third point loading, the same as in beams A & B, was used.

Depth and the covering of the steel

The total depth of the specimens was 6 inches and the covering of the steel was 3/4 inches.

Overall dimensions

Depth = 6"
Width = 9"
Length = 46".

Tension and compression steel

Considerable criticism was voiced against the use of heavy reinforcing steel in beams A & B. Doubtlessly the high diagonal tension stresses sustained by beams A-1 and A-2 were due, in part at least, to the heavy tensile steel. In beams A and B the area of steel in a 12" width = 4.91 sq.inches. For balanced design the area/12" width = 0.92 sq. inches. To keep the area of tensile steel as low as possible and yet attain a diagonal tension stress of at least 0.18 f'_c, it was decided to use compression steel.

Balanced design gave an A_s/12" width of 0.92 sq.inches, a neutral axis at 2.03", a moment of 124,000 in.lbs. and a diagonal tension stress of 283 p.s.i. (see above under Design of Beams A and B). It was decided to increase the diagonal tension stress to 566 p.s.i. which required doubling the moment resisted by the balanced section. To do this, compression steel was added. The covering on the compression steel was 3/4" and the size of the compression steel was 5/8", therefore the distance of the compression steel from the neutral axis was 2.03" - 17/16" = 0.97". With the effective depth of the balanced section as 4.94", the compression force to be taken by the compression steel was 124,000/(4.94 - 1.06) = 32,000 lbs. Allowing for plastic flow, the ultimate stress in the compression steel was taken as 32,000 p.s.i. = 2 x 16,000, in which 2 = safety factor and 16,000 = the design stress given in Sutherland and Rees (1-83). The area of the compression steel = 32,000/32,000 = 1.0 sq.inches/12" width. The area of the tension steel is double since the compression force is doubled, and thus the area of the tension steel = 1.84 sq.inches/12" width. A description of the cross section is given in drawing 16. This section was analysed (1-82) and the predicted failure load for a section 9" wide was 47.5 kips with a diagonal tension stress at failure of 645 p.s.i.

Bond requirements of tension and compression steel

For the tensile steel a stress of 40,000 p.s.i. had to be developed. A standard hook will develop 20,000 p.s.i. and the 15" from the tangent of the hook to the point of maximum stress will develop, according to Prof. Mylrea (12-533), an additional 28,000 p.s.i. A standard hook with an inside diameter of three inches will provide sufficient anchorage for the tensile steel.

No data of ultimate strengths of bond between concrete and compression steel exist. As a safe approximation the bond values for concrete and tension steel were used. To develop a stress of 32,000 p.s.i. in 5/8" bars a length of 18" is required. Thus the compression steel was made to extend the length of the specimen.

Buckling stirrups for the compression steel

The Joint Committee Code states, "compression reinforcement in girders or beams should be secured against buckling by ties or stirrups adequately anchored in the concrete and spaced not more than 16 diameters apart" (6-52). Ties of 1/4" diameter were placed four inches either side of the middle of the beam.

Drawing 17 shows the tension steel, compression steel and the ties.

BEAMS C-1 and C-2

Tension steel

The beams contained no web reinforcement and served as standards. For reasons of economy and without appreciably changing the tensile area, it was decided to use five 5/8" bars instead of three 3/4" bars as the tension reinforcement of these two beams. In all other respects, beams C-1 and C-2 resembled drawings 16 and 17.

Gages

SR-4 gages were placed on the tension steel half way between the support and the load point. Their purpose was to attempt to establish a relationship between the elongation of the tension steel and the formation of diagonal tension cracks. These gages which were all placed at the mid depth of the bars are shown on drawing 19.

Bond gages were used to measure the end slip of the tension bars. Drawing 18 shows how these gages were applied (Fig. 12).

Test data of beam C-1

The positions of the SR-4 gages and the bond gages are given on drawing 19.

Load kips	SR-4 Gages micro-in./in.			Bond Gages 1/1000"	
	1	2	3	A	B
6	5450	6970	6300	9.1	33.0
12	5570	7090	6460	do	do
18	5710	7240	6630	do	do
22	5830	7350	6790	do	do
26	5960	7450	7140	do	do
30	6190	7530	7440	do	30.0
34	6370	7590	7640	do	25.5
38	6480	7640	7730	do	22.0

Test phenomena of beam C-1

Drawing 20 shows the formation of diagonal tension cracks.

Test data of beam C-2

The positions of the SR-4 gages and the bond gages are given on drawing 19.

Load kips	SR-4 Gages micro-in./in.			Bond Gages 1/1000"	
	1	2	3	A	B
0	6930	5900	5980	80.0	28.0
2	6930	5910	5980	do	do
6	6950	5950	6020	do	do
12	6960	5980	6080	do	do
18	6970	6170	6290	do	do
22	6990	6310	6410	do	do
26	7010	6420	6500	do	do
30	7120	6520	6570	do	do
34	7300	6650	6660	do	do
38	7500	6770	6760	79.5	do
42	7770	6860	6800	79.0	27.8
46	7910	6950	6890	78.9	27.5
50	8080	7060	7010	78.0	27.0

Test phenomena of beam C-2

Drawing 21 shows the formation of diagonal tension cracks.

Discussion of results of beams C-1 and C-2

Assuming jd to be 4.08, the ultimate diagonal tension stress of beam C-1 = V/bjd = 20,500/9 x 4.08 = 550 p.s.i. and the ultimate diagonal tension stress of beam C-2 = 26,500/9 x 4.08 = 720 p.s.i.

Diagonal tension haircracks appeared at diagonal tension stresses of 410 and 462 p.s.i. for beams C-1 and C-2 respectively.

The angle of the diagonal tension cracks was about 45 degrees.

The SR-4 gages gave an average tensile stress at failure of 30,000 p.s.i. and an average tensile stress at the formation of diagonal tension haircracks of 20,000 p.s.i.

In beam C-1, end slip coincided with the formation of diagonal tension cracks. No end slip occurred in beam C-1 at the end where no diagonal tension cracks formed. In beam C-2, end slip did not take place until after the formation of diagonal tension cracks.

In beams A-1 and A-2, diagonal tension failure occurred simultaneously with the formation of the first diagonal tension cracks. In beams C-1 and C-2, the failure loads were about 50 percent greater than the loads at which diagonal tension cracks first formed. The negative steel and the hooks doubtlessly increased the failure load in diagonal tension.

BEAMS D-1 and D-2

Beams D-1 and D-2 had diagonal tension reinforcement made of expanded metal. It was considered that this type of web reinforcement might be better than the B-type reinforcement.

Tension Steel

The five - 5/8" bars which were used in beams C-1 and C-2 did not give sufficient spacing for the concrete. Therefore three - 3/4" bars were used as shown in drawings 16 and 17.

Design of the diagonal tension reinforcement

The diagonal tension force to be taken by the web reinforcement was $3 \times 0.06\, f_c' = 0.18\, f_c'$. For 3,000 lb. concrete, this meant that the force per 12" width was 15,300 lbs. (see design of beams A and B). The type of expanded metal which was chosen had a cross sectional area per running foot of 0.61 sq.inches. The tensile stress in the web reinforcement was 15,300/0.61 = 25,000 p.s.i. which was satisfactory.

The size of the openings was 1 1/8" by 2 1/4" and the cross sectional area of an individual strand was 3/16" by 1/8". Assuming only one opening on either side of the neutral axis, the bearing area per running foot for purposes of anchorage by bond = 12 x 1.2 x 2 x 1/8 = 3.6 sq.inches. Thus the bearing stress = 15,300/3.6 = 4,250 p.s.i. which was considered to be permissible.

The expanded metal weighed two pounds per square foot.

Drawing 22 shows the diagonal tension reinforcement for beams D-1 and D-2, it is referred to as the D-type reinforcement.

Gages

One bond gage was mounted for each beam (Drwg. 18). The positions of these gages are shown on drawing 23.

Test phenomena of beam D-1

The test cylinders failed at 59.0 and 62.4 kips giving an average compression stress of 2,150 p.s.i. The design was for 3,000 lb. concrete (see Appendix) and the low strength of the test cylinders was considered to be the result of poor curing.

The specimen was loaded in intervals of four kips and at each interval the bond gage was read and the beam was inspected for diagonal tension cracks.

The bond gage read 51.5 up to 40 kips at which load the bond gage reading was 51.0. No subsequent change in the bond gage occurred.

Drawing 23 shows the cracks which appeared.

Test phenomena of beam D-2

The test cylinders failed at 79.4 and 71.4 kips giving an average compression stress of 2,700 p.s.i.

Unfortunately the bond gage was affected by the deflection of the beam and did not give any indication of the end slip (Drwg. 24).

Drawing 25 shows the cracks which occurred.

Discussion of results of beams D-1 and D-2

Assuming jd to be 4.08, the diagonal tension stress taken by beam D-1 = $V/bjd = 24{,}000/9 \times 4.08 = 655$ p.s.i. and the diagonal tension stress taken by beam D-2 = $22{,}000/9 \times 4.08 = 600$ p.s.i.

Owing to the initial failures in tension and anchorage, no ultimate diagonal tension strength was obtained. The diagonal tension reinforcing prevented the diagonal tension cracks from widening. The failure cracks for both beams occurred outside of the region reinforced by the diagonal tension reinforcement.

Upon breaking up the beams it was obvious that concrete filled the openings in the expanded metal.

BEAMS E-1 and E-2

Beams E-1 and E-2 had diagonal tension reinforcement made of sand-blasted, 1/16" steel plate (Drwg. 22). The tension and compression steel are shown in drawings 16 and 17.

Design of diagonal tension reinforcement

As in the previous beams, the diagonal tension force to be taken by the reinforcement in a 12" width = 15,300 lbs. (see design of beams A and B). The thickness of the plate was 1/16" which gave a tension stress = 15,300 x 16/12 = 20,000 p.s.i. which was well within the permissible. A thinner plate could have been used from a structural viewpoint, but economically a 1/16" plate was desirable as thinner plates cost more.

The diagonal tension force in the steel reinforcement had to be developed by bond. Taking the distance from the neutral axis to the outer edge of the steel plate as 3", the required bond stress = 15,300/2 x 12 x 3 = 210 p.s.i. In order to improve the bond, the plates were sand-blasted. To gain an idea of the bond stress that could be attained, six test specimens and three cylinders were made (Drwg. 26). These test specimens were tested (Fig. 13) with the following failure loads in pounds: 1120, 1900, 1000, 620, 1340, and 2370. The average bond stress was 230 p.s.i. The cylinders failed at an average compression stress of 2,900 p.s.i.

Test data for beam E-1

The cylinders failed at 55.0 and 61.6 kips giving an average compression stress of 2100 p.s.i. This low value was due to inadequate curing of the cylinders. The position of the bond gage, whose results are recorded below, is given in drawing 27.

Load kips	Gage 1/1000"
1	56.8
6	57.6
12	57.8
18	58.0
24	58.0
30	58.0
34	56.0
38	54.0
40	53.0
42	50.0

Test phenomena for beam E-1

Drawing 27 shows the cracks which occurred.

Test data for beam E-2

The cylinders broke at 52.7 and 84.2 kips giving an average compressive stress of 2400 p.s.i. The position of the bond gage, whose results are recorded below, is given in drawing 28.

Load kips	Gage 1/1000"
1	3.1
6	do
12	do
18	3.8
24	do
+4's	do
40	do
42	3.6
44	3.4
48	3.1
50	3.3
52	3.8
as beam failed	1.0

Test phenomena of beam E-2

Drawing 28 shows the cracks which occurred.

Discussion of results of beams E-1 and E-2

Assuming jd to be 4.08, the diagonal tension stress taken by beam E-1 = 21,500/9 x 4.08 = 590 p.s.i. and the diagonal tension stress taken by beam E-2 = 26,000/9 x 4.08 = 700 p.s.i.

Owing to the initial failure in tension, no ultimate diagonal tension strength was obtained. The failure cracks for both beams occurred outside of the region reinforced by the diagonal tension reinforcement.

Although the bond gages were influenced by the deflection of the beams, they qualitatively indicated end slip (Drwg. 32).

Upon breaking up the beams it was seen that the rodding had been insufficient as the concrete was honeycombed under two of the plates at the level of the tensile steel. No indication of bond failure was noticed.

Difficulty was found in placing concrete around this type of reinforcing. For its effectiveness, this diagonal tension reinforcement depends on the "surface" bond between steel and concrete which is inferior to bond by bearing. These two factors, placement of concrete and "surface" bond, militate against the acceptance of this type of reinforcement.

<u>BEAMS F-1 and F-2</u>

Beams F-1 and F-2 had diagonal tension reinforcement identical to that used in beams B-1 and B-2 (Drwg. 22). The design of this reinforcement is given under the design of beams B-1 and B-2 (see previous section). The tension and compression steel are shown in drawings 16 and 17.

<u>Gages</u>

SR-4 strain gages were placed on the stirrups. These gages were the smallest size, designated by A-7, in order to minimize the bond interruption. The positions of the SR-4 gages and the bond gages are shown on drawings 29 and 30 for beams F-1 and F-2 respectively. Section X-X, drwg. 29, which is typical of all the SR-4 gages, gives their position on the web reinforcement.

Test data for beam F-1

The cylinders broke at 50.6 and 78.4 kips giving an average compression stress of 2300 p.s.i. This low strength was caused by inadequate curing of the cylinders. The results of the strain gages and the bond gage are given in the following table (Drwg. 29).

Load kips	SR-4 Gages micro-in./in.		Bond 1/1000"
	A	B	
0	7530	7190	78.1
1	7540	7190	do
4	7530	7190	do
8	7530	7200	do
12	7540	7210	78.5
16	7540	7220	78.8
20	7540	7230	78.9
24	7590	7240	79.0
28	7650	7240	do
30	7690	7240	do
32	7730	7240	do
34	7770	7240	do
36	7820	7240	do
38	7890	7240	78.8
40	7950	7250	78.6
42	7960	7260	78.1
44	8000	7280	77.9
46	8030	7340	77.0
48	8050	7490	76.2
50	8090	7700	75.0
52	8200	7760	69.1

Test phenomena of beam F-1

Drawing 29 shows the cracks which occurred.

Test data for beam F-2

The cylinders broke at 52.6 and 60.6 kips giving an average compression stress of 2200 p.s.i. This low strength was caused by inadequate curing of the cylinders. The results of the strain gages and the bond gage are given in the following table (Drwg. 30).

Load kips	SR-4 Gages micro-in./in.		Bond 1/1000"
	A	B	
0	inoperative	7770	97.1
1		7770	do
4		7780	do
8		7780	do
12		7790	do
16		7800	97.5
20		7810	do
24		7820	do
28		7830	do
30		7840	do
32		7840	do
34		7855	do
36		7870	98.1
38		7890	99.0
40		7910	99.5
42		7950	100.0
44		8010	99.5
46		8080	99.1

Test phenomena of beam F-2

Drawing 30 shows the cracks which occurred.

Discussion of results of beams F-1 and F-2

Assuming jd = 4.08, the diagonal tension stress taken by beam F-1 = 26,500/9 x 4.08 = 720 p.s.i. and the diagonal tension stress taken by beam F-2 = 24,000/9 x 4.08 = 650 p.s.i.

The two stirrups of beam F-1, which had SR-4 gages, were stressed to 20,000 and 17,000 p.s.i. and the stirrup of beam F-2 was stressed to 13,000 p.s.i. (Drwg. 31).

Owing to the initial failure in tension and anchorage, no ultimate diagonal tension strength was obtained.

Although the bond gages were influenced by the deflection of the beam, they qualitatively indicated end slip. A recommended bond gage is shown in drawing 32.

A study of drawings 29, 30 and 31 revealed that two of the stirrups received appreciable stress only upon the formation of diagonal tension cracks. The stirrup which did not act in this manner was affected by the strain of the horizontal tensile steel.

COSTS OF MANUFACTURE OF B, D and E TYPES

All three types of diagonal tension reinforcement were structurally satisfactory. Before deciding which was the best, the cost of manufacture and several other factors (see below) were considered.

In investigating the costs of manufacture, a flat plate building with 10" columns and 6" slabs was considered. The maximum diagonal tension to be resisted was taken as 600 p.s.i., of which the concrete was assumed to take 200 p.s.i. The effective depth was taken as t - 1.5 = 4.5" and the first critical section for diagonal tension was assumed to be at 4.5" from the face of the column (Drwg. 33). The second section for diagonal tension was taken at 9" from the face of the column and the third at 13.5" from the face of the column (Drwg. 33). Reinforcement was placed at the three critical sections which meant that every potential diagonal tension crack would cross diagonal tension reinforcement (Drwg. 33).

B Type reinforcement

First section

The reinforcement was required to take 400 p.s.i. and the force to be carried in a 12" width = 15,300 lbs. A stirrup size of 3/8", rather than a smaller size, was chosen in order to (1) minimize the number of welds, (2) offer minimum resistance to the placement of the concrete. The stirrups and the tie rods were designed as those in beams B-1 and B-2. For a total outside perimeter of 4(10 + 4.5 + 4.5) = 76", the first section required 48 welds, 16' of 3/8" bars and 10' of 1/2" bars.

Second section

The total outside perimeter of the second section was

4(10 + 4.5 + 4.5 + 4.5 + 4.5) = 112", therefore the total diagonal tension stress = 600 x 76/112 = 410 p.s.i., requiring the reinforcement to take 210 p.s.i. The force in the reinforcement for a 12" width = 800 lbs./ft. or approximately one half of the force for the first section. The method of the design of the stirrups was the same as in beams B-1 and B-2; the result was two 3/8" legs per foot instead of the four per foot as in beams B-1 and B-2. Theoretically, the tie rods could have been 1/4" in diameter, but for purposes of rigidity 3/8" tie rods were used. The second section required 29' of 3/8" bar and 40 welds.

Third section

This section, which lay outside the other two, had a perimeter of 4(10 + 4 x 9) = 184". In the same manner as for the second section, the materials were calculated to be 33' of 3/8" bar and 32 welds.

Total cost

Three welding companies of Montreal, Dominion Welding, Keating Foundry and Rudel Machinery, stated that the cost per weld would be about 5 cents. The reinforcing cost 10 cents per pound.

welding	- 120 x 5	=	6.00
reinforcing	- 3/8"	=	2.94
	- 1/2"	=	67
			$9.61

The total cost of the B-type of reinforcement = $9.60 per column.

D Type reinforcement

First section

The steel stress was 400 p.s.i. and expanded metal, weighing two pounds per square foot, was used. The design was the same as for beams D-1 and D-2, and with a cross sectional area of 0.61 sq.inches the tensile stress was 15,300/0.61 = 25,000 p.s.i. The area of expanded metal = 4 x 19 x 8/12 x 12 = 4.2 sq.ft.

The cost of this expanded metal was 35 cents per square foot according to the Pedlar People Limited. For cutting and placing, a cost of 10 cents per square foot was assumed. The cost for the first section = 4.2 x 45 = $1.90.

Second section

For this section an expanded metal, weighing 0.8 lbs. per sq.ft. was used. The tensile stress was 34,000 p.s.i. and the area = 6.2 sq.ft.

The cost of material was 15 cents per square foot plus 10 cents for cutting and placing. The cost for the second section = 6.2 x 25 = $1.60.

Third section

For this section an expanded metal weighing 0.6 lbs. per sq.ft. was used. The tensile stress was 23,000 p.s.i. and the area = 8.2 sq.ft. The cost of the third section = 8.2 (11 + 10) = $1.80.

Total cost

The total cost for the D type reinforcement was $5.30.

E Type reinforcement

The least expensive thin plate was 1/16" in thickness. This thickness was considered for all three sections. The tension stress for the first section = 20,000 p.s.i., and the area of plate = the area of expanded metal = 18.6 sq.ft.

The cost of the plate = 15 cents per sq.ft., the cost of sand-blasting per sq.ft. = 20 cents and the cost of cutting and placing = 10 cents. The total cost of the three sections of E type reinforcement = $8.30.

Summary of Costs

B Type - $9.60

D Type - $5.30

E Type - $8.30.

CHOICE OF DIAGONAL TENSION REINFORCEMENT

The five factors considered in this choice were: structural strength, certainty of design, cost of manufacture, job problems, and adaptability to laboratory experiments.

The beam tests indicated that all three types were satisfactory for structural strength.

The relative certainty in design of the three types of reinforcement was in the following order: B, D and E. In the B type, both the tensile stress and the bond could be designed for. In the D type, although the tensile stress could be designed, the bond by bearing could only be approximated. In the E type, the tensile stress could be designed but the "surface" bond was not as reliable as the bond by bearing.

The cost of manufacture showed that the D type was 55 percent of the cost of the B type and 64 percent of the cost of the E type.

The main job problems were those of placing the reinforcement around the column and of pouring the concrete. On both these counts the B type was greatly to be preferred because of (1) the rigidity of each section and (2) the size of its openings. Difficulty was experienced in placing concrete around the E type.

In order to further investigate the problem in the laboratory it was desirable to place SR-4 gages on the diagonal tension reinforcement. Types B and E were suitable for this, type D was not.

After a consideration of the above factors, type B was chosen for use in the subsequent slab specimens.

SLABS G, H, I and J

PURPOSE

These specimens were made in order to experiment with the B type reinforcement in flat plate construction and represented the portion of a flat plate building around the top of a column.

DESIGN

Number and general description of specimens

Four slab specimens were made, of which slab G was the standard with no diagonal tension reinforcement. Slabs H and I had B type reinforcement whose sections were square in plan (Drwg. 33), and slab I had B type reinforcement whose sections were circular in plan in order that the circular tie rods would take the circumferential stresses.

All slab specimens were loaded on their column stubs and were continuously supported along their outer edges (Drwg. 34 and Fig. 15).

Overall dimensions

In designing the overall dimensions, three governing factors were considered. (1) The testing apparatus was limited to 150 kips. (2) The desired maximum diagonal tension stress = $3 \times 0.06 f'_c$ = 540 p.s.i. for 3,000 lb. concrete. (3) A study of Mr. Gersovitz's tests of slab specimens similar to the ones under consideration showed that excessive bending moment stresses occurred at a load of 130 kips (4-63 and table 6).

The critical section for diagonal tension was assumed to be at a distance from the face of the column equal to the distance from the top of the slab to the tension steel. Assuming a concrete covering of 3/4" and two layers of 5/8" bars, this distance = 6 - 3/4 - 5/8 - 5/16 = 4.37". For a 12" square column b = 4(12 + 2 x 4.37) = 83 inches and assuming jd to be 3.4 inches the diagonal tension stress for maximum load = 150,000/83 x 3.4 = 534 p.s.i. To increase the diagonal tension stress the column size was taken as 10" which gave a diagonal tension stress of 590 p.s.i. The height of the column was 1' - 2" in order to fit the testing apparatus.

The depth of the slab was 6" and the effective width was taken as 3' - 0" square. The width was purposely made less than that used by Mr. Gersovitz in order that a load of 150 kips might be attained without exceeding the permissible bending moment stresses. As will be seen below, this width of 3' - 0" was correct for the design of the tensile steel and on the safe side for the design of the compression concrete.

The overall dimensions of the specimens were: depth = 6", effective width = 36" square, total width = 44" square, the concentric column stub = 10" square x 14" high (Drwg. 34).

<u>Tension and column steel</u>

In calculating the resisting moment, the moment was assumed to be equally distributed across the effective width of the slab. The load was assumed to be equally distributed along the supporting edges.

The load per foot along the supporting edges = 150,000/4 x 3 = 12,500 lbs./ft. The moment at the face of the column per foot width = (36/2 - 10/2) x 12,500 = 162,000 in.lbs. The resisting moment of the concrete, according to the cubic formula, = $0.346 \times b \times d^2 \times f'_c$ = $0.346 \times 12 \times 4.37^2 \times 3{,}000$ = 238,000 in.lbs. Assuming the neutral axis to be 3" from the top, thereby making jd = 3.37", and considering 5/8" round bars at 3" centers having a steel area of 1.24 sq.in./ft., the moment resisted by the tensile steel = 40,000 x 3.37 x 1.24 = 167,000 in.lbs. Theoretically the design was adequate for bending moment.

To help in developing a tensile stress of 40,000 p.s.i., a standard hook was used. The hook plus the 13" of straight bar were capable of developing 48,000 p.s.i. (12-533).

The ultimate strength of a reinforced concrete column can be taken as 0.85 f'_c x area of the concrete plus f'_s x area of the steel (1-114). The ultimate strength provided by the concrete of a 10" column = 100 x 0.85 x 3,000 = 255,000 lbs. Theoretically, no reinforcement was required. As a safety factor, four 5/8" bars with two 1/4" ties were used.

SLAB G

This slab was the standard for the other three and contained no diagonal tension reinforcement. Its design is given above and drawing 34 shows the details of this slab (Fig. 14).

Gages

Three Ames dial deflection gages were used and their positions are shown on drawing 35.

Loading

Figures 15, 16, 17 and 18 show the slab specimens in the testing apparatus. The loading block was made of steel, was equipped with four SR-4 strain gages and was well within the elastic range for all loads. An exhaustive description of this loading block is given in Mr. Gersovitz's thesis (4). The writer recommended the purchase of an aluminium loading block because it would be more sensitive than the steel one due to the low modulus of elasticity of aluminium.

The loading block was calibrated in a testing machine immediately before the slab was tested. This calibration related the strain gage readings with the applied load. The loading block was then placed on top of the slab (Fig. 18) and the load was applied by means of the hydraulic pump (Fig. 17). The load was calculated from a knowledge of the strain gage readings and of the calibration results.

Test data

The two cylinders which were tested prior to testing the slab failed at 79.0 and 86.0 kips; the two cylinders which were tested after testing the slab failed at 87.6 and 82.5 kips. The average compression stress was 3,000 p.s.i.

The following table records the readings of the deflection gages (Drwg. 35).

Load kips	Deflection Gages 1/1000"		
	1	2	3
0	26	31	35
8.7	39	55	63
18	50	72	80
27	58	85	94
34	65	93	103
42	72	101	112
52	82	115	125
61.5	91	128	137
70	102	142	149
78	110	152	160
84	115	160	168
90	122	172	179
96	130	182	190
106	140	198	209
109	195	280	308

Test phenomena

Drawing 36 shows the cracks which formed.

The failure was sudden, and of especial significance was the fact that the tension cracks did not widen subsequent to 70 kips (Drwg. 36).

Drawing 37 shows the cracks that were found upon breaking up the slab. Figures 19 and 20 help one to visualize the nature of the failure.

Discussion of results

The specimen failed suddenly in diagonal tension. With the effective depth = 6 - 3/4 - 5/8 - 5/16 = 4.37" and assuming the critical section for diagonal tension to occur at a distance of 4.37" from the face of the column, then b = 4(10 + 2 x 4.37) = 75" and the diagonal tension stress at failure = 109,000/75 x 0.875 x 4.37 = 380 p.s.i. (jd = 0.875 x d = 0.875 x 4.37").

The angle of failure was approximately 45 degrees.

SLAB H

This slab was identical to slab G except that B type diagonal tension reinforcement was used.

Design of diagonal tension reinforcement (Fig. 21 & 22)

The maximum diagonal tension stress was assumed to be 600 p.s.i. and the concrete was assumed to take 200 p.s.i. leaving 400 p.s.i. for the steel.

The reinforcement was of the B type and was placed in squares around the column (Drwg. 38) (Fig. 21 & 22). The sections, as the four sides of one square were called, were not welded at the corners and were spaced t - 1.5 = 4.5" apart.

The first section was placed so that its tension hoops were in the same vertical plane as the column face (Drwg. 38). This position was different from that suggested by the Portland Cement Association in R/C 22. In R/C 22, the first section was 2 3/4" from the column face (5-3).

In designing the horizontal tie rods, two factors were considered: (1) that the concrete covering for the steel was 3/4" and (2) that both tension and compression grids of 5/8" bars had to be allowed for. These factors meant that the tie rods were placed a vertical distance of 3/4 + 5/8 + 5/8 = 2" from both the top and the bottom of the slab.

First section (Drwg. 38)

The diagonal tension stress to be taken by the steel was assumed to be 400 p.s.i. This caused a total force per foot of 15,300 lbs. and, as in the design of beam B, three 3/8" stirrups and two 1/2" tie rods were used for each side (Drwg. 38).

Second section

The diagonal tension stress on this section was equal to the diagonal tension stress on the first section times the width of the first section divided by the width of the second section = 600 x 4(10 + 2 x 4.5)/4(10 + 2 x 9) = 410 p.s.i. The concrete was assumed to take 200 p.s.i. leaving the steel with 210 p.s.i. or 800 lbs. per foot width. The total force per side remained the same as for the first section, thus three 3/8" stirrups were required. Theoretically, 1/4" tie rods would have sufficed but 3/8" tie rods were used in order to increase the rigidity (Drwg. 38).

Third section

By the method used for the second section, the diagonal tension stress in the steel = 120 p.s.i. which necessitated two stirrups per side (Drwg. 38).

<u>Gages (Fig. 21 & 22)</u>

Six SR-4 gages were applied to the diagonal tension reinforcement. These gages were of the ordinary A-12 type and completely interrupted the bond between the tie rods. Because of this interruption the gage readings could at best only be considered as indications of the stress taken by the diagonal tension reinforcement. Drawing 38 shows the positions of the SR-4 gages.

As in slab G, three deflection gages were used (Drwg. 35).

Test data

The four cylinders broke at 88.3, 84.1, 112.0 and 106.9 kips giving an average compression stress of 3400 p.s.i.

The following table records the gage readings. (Drwg. 39).

Load kips	Deflection Gages 1/1000"			SR-4 Gages micro-in./in.					
	1	2	3	1	2	3	4	5	6
0	210	250	057	6800	7010	7580	6830	7170	7360
12	253	304	098	6810	7030	7590	6850	7200	7370
22	280	333	122	6830	7040	7590	6880	7270	7390
37	316	371	146	6840	7050	7590	6940	7390	7440
52	336	394	164	6850	7060	7590	6970	7460	7470
64	353	413	179	6870	7080	7600	7010	7540	7510
74	364	427	189	6880	7100	7600	7030	7590	7540
85	378	437	205	6880	7110	7600	7090	7670	7600
95	392	461	213	6890	7090	7630	7150	7670	7650
105	430	500	240	6890	6910	7800	7300	7600	7730
110					4990	8720		7760	8060

Test phenomena

Drawing 40 shows the cracks which formed during the test and shows the cracks that were found upon breaking up the slab. No cracks appeared to cross between the ties of the diagonal tension reinforcement.

Discussion of results

The specimen was considered to have failed initially in tension and secondly in compression, although there was no convincing evidence that this order of failure was not reversed (i.e. compression failure first).

The results of breaking up the specimen and the observed cracks on the bottom surface showed that the reinforcement prevented the formation of diagonal tension cracks. The maximum nominal diagonal tension stress successfully taken by the reinforcement = 110,000/75 x 0.875 x 4.37 = 384 p.s.i. (i.e. taken by the reinforcement plus the concrete).

SLAB I

This slab was identical to slab H in all respects. (Drwg. 38, Fig. 21 and 22).

Gages

Six SR-4 gages were applied to the diagonal tension reinforcement. These gages were of the ordinary A-12 type and completely interrupted the bond between the tie rods. Because of this interruption the gage readings could at best only be considered as indications of the stress taken by the diagonal tension reinforcement. Drawing 41 shows the positions of the SR-4 gages.

As in slab G, three deflection gages were used (Drwg. 35).

Test data

The four cylinders broke at 84.5, 86.0, 89.5, and 83.6 kips giving an average compression stress of 3,000 p.s.i.

The following table records the gage readings. (Drwg. 43).

Load kips	Deflection Gages 1/1000"			SR-4 Gages micro-in./in.					
	1	2	3	1	2	3	4	5	6
0	221	191	145	2730	inoperative	4146	3330	4060	0420
9	270	240	188	2750		4150	3350	4050	0410
24	300	272	215	2800		4180	3370	4060	0400
36	319	295	235	2820		4220	3480	4070	0400
47	338	318	255	2830		4260	3580	4070	0390
60	361	347	279	2830		4310	3610	4100	0380
72	389	381	308	2820		4370	3680	4130	0400
87	432	432	345	2850		4440	3790	4180	0390
93	465	467	368	2890		4350	3940	4150	0330
100	511	575	422	3340			3950	4120	0430

Test phenomena

Drawing 44 shows the cracks which formed during the test and the cracks that were found upon breaking up the slab. No cracks appeared to cross between the ties of the diagonal tension reinforcement.

Discussion of results

The specimen was considered to have failed initially in compression and secondly in tension, although there was no convincing evidence that this order of failure was not reversed (i.e. tension failure first).

The results of breaking up the specimen and the observed cracks on the bottom surface showed that the reinforcement prevented the formation of diagonal tension cracks. The maximum nominal diagonal tension stress successfully taken by the reinforcement = 100,000/75 x 0.875 x 4.37 = 350 p.s.i.

SLAB J

Slab J was identical to slab G except that B type reinforcement with circular tie rods was used. These tie rods were continuous and were intended to minimize cracking along the diagonals (Drwg. 36, 40 and 44).

Design of diagonal tension reinforcement (Drwg. 42, Fig. 23)

The maximum diagonal tension stress was assumed to be 600 p.s.i. and the concrete was assumed to take 200 p.s.i.

As in slabs H and I, the first section was placed so that its tension hoops were in the same vertical plane as the column face and the tie rods were placed a vertical distance of 2" from both the top and the bottom of the slab. The sections were spaced t - 1.5" = 4.5" apart. (Drwg. 42).

First section

The first section was assumed to be a circle whose diameter equaled 2 x 4.5" = 9" plus the diameter of a circle equal in area to the area of the column. The diameter of the first section = 9 + 100 x 4/3.14 =

20.3". As in the design of beam B, the stirrups were designed to take 15,300 lbs. per foot which required two 3/8" stirrups per foot. The circumference of the first section = 3.14 x 20.3 = 63.8" or 6 feet which necessitated 12 stirrups.

In the first section the circular tie rods were 1/2" in diameter.

Second section

The diagonal tension stress at the second section = 600 x 20.3/29.3 = 410 p.s.i. which caused a diagonal tension stress in the steel of 210 p.s.i. This stress required one stirrup per foot width and as the circumference = 3.14 x 29.3 = 92" or 8 feet, therefore 8 stirrups were used.

Theoretically, 1/4" tie rods could have been used, but for purposes of rigidity 3/8" tie rods were employed.

Third section

This section was designed in the same manner as the second section; the design resulted in five stirrups and 3/8" tie rods.

Gages (Fig. 23)

Six SR-4 gages were applied to the diagonal tension reinforcement. These gages were of the ordinary A-12 type and completely interrupted the bond between the tie rods. Because of this interruption the gage readings could at best only be considered as indications of the atress taken by the diagonal tension reinforcement. Drawing 42 shows the positions of the SR-4 gages.

As in slab G, three deflection gages were used (Drwg. 35).

Test data

The four cylinders broke at 82.6, 70.9, 66.7 and 81.5 kips giving an average compression stress of 2700 p.s.i.

The following table records the gage readings. (Drwg. 43).

Load kips	Deflection Gages 1/1000"			SR-4 Gages micro-in./in.					
	1	2	3	1	2	3	4	5	6
0	155	192	141	inoperative	6660	8160	inoperative	6900	4600
16	222	278	220		6630	8090		6810	4510
30	260	312	259		6590	8230		6900	4550
37	274	328	272		6660	8310		6950	4600
43	285	339	282		6750	8350		6970	4600
70	358	406	330			8450		6990	4510
75	372	418	338					6930	4400
85	490	498	386					7300	5240

Test phenomena

Drawing 45 shows the cracks which formed during the test and the cracks that were found upon breaking up the slab. No cracks appeared to cross between the ties of the diagonal tension reinforcement.

Discussion of results

The specimen was considered to have failed initially in compression.

The SR-4 gages did not act in their normal manner. No reason for their poor behaviour was apparent as the gages were tested and found satisfactory before being applied.

The results of breaking up the specimen and the observed cracks on the bottom surface showed that the reinforcement prevented the formation of diagonal tension cracks. The maximum nominal diagonal tension stress successfully taken by the reinforcement = 85,000/75 x 0.875 x 4.37 = 300 p.s.i.

CONCLUSIONS OF SLABS G, H, I and J

The slab without diagonal tension reinforcement failed in diagonal tension at a stress of 380 p.s.i. The slabs with the B type reinforcement did not fail in diagonal tension; the B type reinforcement was effective in taking diagonal tension stresses of 300, 350 and 384 p.s.i. No ultimate test of the diagonal tension reinforcement was possible as the slabs failed in bending moment.

About 40 percent of the SR-4 gages showed stresses of about 20,000 p.s.i. at maximum loads thus indicating that the reinforcement was being subjected to considerable stress. Because the gages interrupted the bond, too much significance should not be given to the SR-4 readings.

A study of the deflection gage readings did not give any conclusive information.

Slabs H, I and J failed in bending moment at loads equal to about two thirds of the calculated failure load in bending moment. The beam experiments showed that the formulae for predicting beam failure are reliable and, if anything, conservative. Therefore the discrepancy between the computed and actual failure loads for the slabs was not the result of faulty formulae but rather was the result of incorrect design assumptions. The incorrect assumptions were: (1) that the load was equally distributed around the supporting edges of the slab, and (2) that the bending moment was equally distributed across the width of the slab.

This discrepancy indicates the danger in the design of flat plate buildings of assuming that the full width of the column strip may be considered as effective in resisting compression (10-6 & 8). In flat plate construction, as in the test slabs, the moment concentration around the column must be great. However two factors of the flat plate construction tend to lessen this moment concentration. The first factor is the continuous nature of flat plate construction which causes the distribution of bending moments and counteracts the concentration of bending moments. The second factor is that the load for flat plate construction is assumed to be uniformly distributed. This reduces the moment which accompanies a given shear stress. If a uniformly distributed reaction had been used in the test slabs, instead of the reactions which were concentrated around the edge, the moment for a given shear would have been halved. Despite these two factors, the assumption that the full column strip is effective appears to be questionable. This critical problem is being studied at the University of Toronto, during 1949, by Mr. Keith Ebbern, graduate student in Civil Engineering.

In footings the reaction of the earth pressure is normally assumed as uniformly distributed. The second factor mentioned above applies to the problem of footing design. Prof. Richart, in his tests on column footings, found that the bending moment was evenly distributed across the section (11-112). In his tests the specimens were supported on car springs in order to simulate uniformly distributed load. He concluded, "It seems reasonable from the tests to consider all bars crossing the section as fully effective in resisting moment." (11-117). In any future work on the problem of this thesis, the load at which bending moment failure occurs should be increased. This would permit the diagonal tension reinforcement to be tested to higher stresses. One way in which to increase the load at which bending moment failure occurs is by using spring reactions instead of an edge support. If springs were used the bending moment for a given load would be halved and the moment would be approximately evenly distributed across the section.

CONCLUSIONS OF EXPERIMENTS

In the 6" beam tests, all three types of diagonal tension reinforcement were found to be effective in resisting stresses up to 0.2 f'_c. No ultimate strengths of the reinforcement were obtained because of initial failure in bending moment.

The gages used in the beam tests indicated the relationship between diagonal tension, bond and the elongation of the tensile steel.

In the slab specimens the most suitable type of diagonal tension reinforcement was used. The reinforcement was found to be effective in resisting stresses up to 0.12 f'_c. No ultimate strengths of the reinforcement were obtained because of initial failure in bending moment.

The actual bending moment failure loads in the slab specimens were only two thirds of the computed failure loads. This discrepancy indicates the danger in the design of flat plate buildings of assuming that the full width of the column strip is effective in resisting compression.

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APPENDIX

DESIGN OF THE MIX

INTRODUCTION

Strength and workability were the qualities for which the mix was designed. Durability was not sought after. It is a well known fact that modern cements will give high strength concrete without providing correspondingly high durability. The following appendix does not hold for the design of plain concrete where durability must be attained. In this work the concrete was designed for strength and workability.

The materials were investigated as follows: cement for strength, sand for organic impurities, the sand and stone for cleanness.

In the hope of obtaining 3,000 lb. concrete of good workability, three mix "recipes" were tried. Nine test cylinders were made, three per mix. The results of the recipes were not satisfactory.

When the entire three recipes were found unsatisfactory the bibliography given at the end of this appendix was studied. From this study certain fundamentals of mix design were extracted. These fundamentals were the basis of the procedure for mix design which gave satisfactory results.

The only difficulty encountered was that of determining the water content. The American Concrete Institute (6-100) states "A few batches of concrete are mixed on the job and the average quantity of water added for the desired slump is determined." This recommendation did not prove satisfactory, and in point 9 of the procedure of this appendix a more specific method is given for determining the water content.

The result of the procedure was a job curve which relates the cement content per cu.yd. and the compressive strength for a constant water content. From this job curve the design mix was taken.

INVESTIGATION OF MATERIALS

The investigation of the materials was not extensive as the mix was designed only for strength and workability. Where durability is involved a much more thorough investigation must be made. A list of récommended tests for the thorough investigation is given by the American Concrete Institute (6-110).

The strength of the cement was determined by the method recommended by the Canadian Standards Association. (5-37 & 22).

The normal consistency was obtained from the following table which relates the water percentage and the penetration of the Vicat needle in one half a minute.

Percent Water	Penetration m.m. in 1/2 min.
25	6
28	4
30	7
40	40
35	25
30	17
25	5
27	5
28	7
29	7
30	10

For normal consistency a water percentage of 30 was used.

Six briquettes were made from 200 gms. of cement, 600 gms. of sand and 92 c.c. of water. The tensile strengths of three briquettes after a one day interval were 290 gms., 225 gms., and 265 gms. These values compare favourably with the required strength of 275 gms. (5-37). The tensile strengths of three briquettes after a three day interval were 440 gms., 440 gms., and 295 gms. These values compare favourably with the required 3 day strength of 375 gms. (5-37).

In testing the sand for organic impurities, the procedure given by the Portland Cement Association was followed (2-23). The specimens were compared with the standard colour charts (2-57) and the results were satisfactory.

The cleanness tests that were carried out for the sand and the stone are outlined by the Portland Cement Association (2-57). The results of these tests were satisfactory in that the sand had 1 per cent finer than the 200 sieve and the stone had 0.6 per cent. The maximum percentages are 3 and 1 for sand and stone respectively.

RECIPES FOR MIX DESIGN

Numerous tables and graphs of mix recipes exist. Three recipes were tried which were supposed to give workable, 3,000 lb. concrete.

For each recipe, as in all the work of the design of the mix, the following materials were used. High Early Cement, 1/2" Crushed limestone and Charette sand and tap water. A mix of approximately three quarters a cubic foot was made for each recipe. The order of mixing was: stone, sand, cement and water. The mixer had a capacity of 3 cubic feet and ran at 24 revolutions per minute. The materials were mixed for three minutes after the water was added. A slump test was taken and the materials were re-mixed for one minute before the cylinders were poured. Three cylinders, 6" diam. and 12" high, were poured for each mix. The cylinders were cured by being immersed in water and were tested after 7 days .

The first recipe was the one which Mr. Gersovitz used in doing his graduate work at McGill University in 1947. The second recipe was taken from the design graph of the Canada Cement Company and the third was taken from a table of mix designs of Prof. Dodd.

The following table contains the data relevant to the mix "recipes".

Mix	No.1	No.2	No.3
Cement, lbs.	10.6	14.1	14.6
Sand, lbs.	35.0	43.0	45.4
Stone, lbs.	43.0	55.0	51.4
Water, lbs.	7.4	9.3	10.7
Water cement ratio (by weight)	0.70	0.66	0.73
Slump, in.	3	2	3
Workability	fair	fair	excellent
Cohesion	poor	poor	excellent
Surface	honey-combed	honey-combed	excellent
Compressive strength p.s.i.	2330	4000	3820
	3470	2100	3780
	3840	3980	4100

From the above table it is obvious that none of the recipes was able to produce workable, 3,000 lb. concrete. Upon the failure of any of the recipes to provide the desired concrete, the bibliography given at the end of this appendix was studied. From this study certain fundamentals of mix design were culled. These fundamentals are given in the next section.

FUNDAMENTALS OF MIX DESIGN

Introduction

Laboratory tests should be made where strength requirements are of most importance (6-105). A representative and convenient size of batch for a trial mix is one containing around 10 lbs. of cement. The precision of concrete compression tests is about 10%.

For commercial work a strength of 3,450 lbs. is specified for preliminary laboratory tests for 3,000 lb. concrete (4-23). At least 90% of the test specimens shall exceed the specified strength (3,450 lbs.) and the balance shall not fall short of the requirements by more than 10 per cent.

A job curve should be made relating either the water-cement ratio or the cement content with the strength (7-709). This curve is to be made by at least 3 points, each point representing the average value from at least four test cylinders. (2-47).

General

The water-cement ratio law for mixes of proper consistency is the basis of strength design. Although such properties as durability and watertightness are increased with an increase in strength, it is possible to attain strength requirements and yet be deficient in durability and watertightness. For exposed structures the water cement ratio should be obtained from tables relating the water cement ratio to the type of structure and the degree of exposure (6-95). The Joint Committee states "for concrete not exposed to the weather such as interiors of buildings and portions of structures entirely below ground, no exposure hazard is involved, and the water-cement ratio should be selected on the basis of the strength and workability requirements (6-95)."

A low water content is highly desirable (1-126). Wet pastes cause laitance which is a scum containing cement particles and which therefore reduces strength (3-25). The initial water content is the principal contributor to the voids in the concrete; too much water causes a decrease in durability and watertightness and an increase in permeability (1-24). Wetness increases segregation of the coarse aggregate from the mortar and also increases the shrinkage (1-26 & 48). Excessive water is uneconomical since it means excessive cement for a constant water-cement ratio. Too little water causes poor workability and honey-combing.

The water content increases with an increase in the percentage of sand. However when the sand content is low the concrete is difficult to finish, it separates easily and results in a honey-combed surface. A honey-combed surface permits the entrance of water and therefore seriously decreases the durability of the concrete (9-11). The test results of the "recipe" mixes indicate that honeycombed concrete is unreliable for strength.

The slump can be kept the same when the water cement ratio is changed, by maintaining a constant water content and by interchanging the sand and the cement keeping the sum of their absolute volumes the same. (6-101).

Plasticity is indicated by the bulging of the slump specimen and cohesiveness by the specimen's failure to break or crumble when tapped by the tamping rod (1-260).

Within an accuracy of 2% the volume of the concrete equals the sum of the absolute volumes of the constituents (8).

Reductions in strength of as much as 40% have been caused by particle interference. Such reductions are usually found where the coarse aggregate is greater than 2" (10-15). Remedies for particle interference are:

(1) to increase sand percentage

(2) to eliminate interfering particles by sieving.

(3) to proportion mix for a higher strength.

PROCEDURE

Introduction

The following procedure of the design of a mix is based upon the fundamentals already mentioned.

An alternative to the following procedure is the fineness modulus method (11). The fineness modulus method of design is based upon an empirical factor, the fineness modulus, and upon the happy discovery that if the cement is included in the modulus a constant value of the modulus is obtained for all workable mixes from lean to rich.

Although the fineness modulus method was not adopted, the following procedure makes use of the fineness modulus in two ways. Firstly the particle interference is considered, and secondly the trial sand percentage is based on the fineness modulus of the sand (1-139).

Steps

(1) Obtain the specific gravities of the cement, sand and stone (2-62 & 63).

(2) Obtain the percentages of absorption of the sand and stone (2-62 & 63).

(3) Obtain the moisture content of the sand and stone.

(4) Obtain the grading of the sand and stone (2-58).

(5) Obtain the approximate percentage of fine aggregate from Table 17 in the Concrete Manual (1-139).

(6) Choose an approximate water cement ratio (2-40).

(7) Select a unit water content from (a) Table 17, (b) from other tables or graphs, or (c) from past experience.

(8) Knowing the water-cement ratio, the water content and the percentage of sand, calculate the mix proportions (6-100).

(9) Mix up at least three separate batches using the proportions of point 8 and add water until the proper consistency is obtained in each case. The water contents for the three batches should agree within 20 lbs./cu.yd. or the step should be repeated (8-448). A batch size having 10 lbs. of cement is sufficient. From these batches the average water content can be obtained. In obtaining the water content, the mixer and the slump frustrum should be thoroughly clean.

(10) Compute the batch quantities using the water content as found in point 9, the sand percentage in point 5, and the water cement ratio as found in point 6 (6-98).

(11) Compute whether or not particle interference could occur (10-13).

(12) If particle interference does not occur, determine the optimum percentage of sand. The percentage of sand obtained from Table 17 (1-139) was selected

because it is supposed to produce an over-sanded mix (1-150). Decrease the percentage of sand by decrements of 4 per cent. Add water to give the desired consistency. It should be found that for constant slump the unit water content per cubic yard decreases by 2.5 lbs. for each one per cent decrease in the sand (1-150). As the over sanded condition is reduced there is usually little change in concrete workability until a point is reached beyond which harshness and the tendency for segregation become increasingly apparent. This turning point marks the ideal percentage of sand. The practical optimum is one or two per cent higher. The water content for the practical optimum is obtained by interpolation between the water contents for the two closest sand percentages. Adjust the mix to maintain the correct water-cement ratio (6-101). Check the unit water content by preparing three separate batches, each batch having 10 lbs. of cement. Check for particle interference.

(13) Obtain the job curve, which relates the w/c ratio and the strength, by making three batches of sufficient size to fill four cylinders per batch. For these mixes vary the water cement ratio by weight over a range of at least 0.20 to establish the relationship between the water cement ratio and the strength. A range of 0.20 in the water cement ratio by weight gives a range of about 1500 p.s.i. (6-109). It has been found that the strength does not increase with a decrease in the water cement ratio below a value of 0.45 even though the resulting concrete may be workable (8-444).

For the three mixes the water content is maintained constant thereby assuring constant slump. The cement and sand are interchanged by absolute volumes keeping the sum of their absolute volumes constant (6-101). For checking purposes a change in cement of 90 lbs. per cu.yd. will result in a change in sand of about 2% (6-101).

DESIGN OF MIX

Note: the numerical order is the same as for the above procedure.

(1) Specific gravities

Cement (assumed) = 3.15

Stone: Apparent speciric gravity = A/(A-C), in which A = wt. in gms. of oven dry sample in air, B = wt. in gms. of saturated surface dry sample in air, C = wt. in gms. of saturated surface dry sample in water. A = 1620 and 1955, B = 1626 and 1963, C = 1019 and 1235. The apparent specific gravities = 1620/(1620-1019) = 2.69 and 1955/(1955-1235) = 2.70. Therefore the apparent specific gravity = 2.70.

Sand: Apparent specific gravity = A/((500-W) - (500-A)), in which A = wt. in gms. of dry sample in air, V = vol. in ml. of flask, W = wt. in gms. of the water added to the flask. A = 495.2 and 496.0, V = 500.0, W = 314.5 and 313.5. The apparent specific gravities = 2.72 and 2.70. The apparent specific gravity was taken as 2.70.

(2) Percentage absorption

Stone: The percentage absorption = (B-A)/A = (1626-1620)/1620 = 0.37 and (1963-1955)/1965 = 0.41. Therefore the percentage absorption = 0.39.

Sand: The percentage absorption = (500-A)/A = (500-495.2)/495.2 = 0.39 and (500-496.0)/496.0 = 0.40. The percentage absorption of the sand = 0.40.

(3) Moisture content

The moisture content = (W-W')/W', in which W = wt. of sample in ordinary condition and W' = wt. of sample after drying.

Stone = (2112-2112)/2112 = 0 moisture content. Sand = (1405-1402)/1402 = 0.21% moisture content.

(4) Sieve analysis of the sand

Size	Weight Retained gms.	Volume Rodded c.c.	% Passing	% Coarser	Cum. % Coarser	Cum. % Retained
4	164	94	84	16	16	16
8	138	82	84	0	16	30
14	185	108	75	9	25	48
28	322	198	38	37	62	80x
48	142	82	26	12	74	94
100	44	22	10	16	90	98
200	0					
Pan	4					
Total	1000				283	

Note: asterisk indicates where grading exceeds that recommended in the grading chart of the A.C.I. (6-107).

Sieve analysis of the stone

Size inches	Weight Retained gms.	Volume Rodded c.c.	% Retained	Cum. % Retained
1 1/2	0			
3/4	1037	688	41	41
3/8	1225	888	49	90x
1/4	212	138	8	98
Pan	26			
Total	2500			

(5) Approximate sand percentage

Table 17 of the Concrete Manual of the U.S. Bureau of Reclamation was used in obtaining the approximate sand percentage. For angular coarse aggregate having a maximum size of 1" and a sand fineness modulus of 2.60 to 2.90, a sand percentage of 46 by absolute volumes is recommended. For the same coarse aggregate but a sand fineness modulus of 2.90 to 3.20, a sand percentage of 49 by absolute volumes is recommended. A compromise value of 48 per cent was chosen. A check on this value was provided by Recipe No.3 (see above). In this recipe a sand percentage of 47 was used. The concrete from this recipe was neither over sanded nor harsh.

(6) Approximate water cement ratio

For the first approximation, a water cement ratio was determined for a concrete strength of 3500 p.s.i. Recipe No.3 had a w/c ratio of 0.72 and provided 3,900 lb. concrete. The P.C.A.'s mix design graph (2-40) indicated that for a strength decrease of 400 p.s.i., the w/c ratio should be increased by 0.04. For 3,500 lb. concrete, an approximate water cement ratio of 0.76 was chosen.

(7) Approximate water content

The Concrete Manual (1-139) suggested an approximate water content of 325 lbs. per cu.yd. Recipe No.3, which provided suitable consistency, contained 353 lbs./cu.yd. A compromise between these two values of 330 lbs./cu.yd. was chosen as an approximate unit water content.

(8) Calculation of mix proportions using approximate values

Water cement ratio = 0.76 by weight and the cement content = 330/0.76 = 434 lbs./cu.yd. The absolute volume of the water and cement = 330/62.5+434/3.15x62.5 = 7.48 cu.ft. The absolute volume of the total aggregate = 27.0-7.48 = 19.5 cu.ft. Hence the absolute volume of the sand = 0.48x19.5 = 9.37 cu.ft. and the absolute volume of the stone = 19.5-9.37 = 10.15 cu.ft. The sand content = 9.37x2.70x62.5 = 1580 lbs./cu.yd. and the stone content = 10.15x2.70x62.5 = 1715 lbs./ cu.yd. The proportions by weight are 434/434:1580/434 : 1715/434 = 1 : 3.64 : 3.96. It is to be noted that the moisture content and the absorption of the aggregates, which are small, have been disregarded in this calculation of preliminary mix proportions.

(9) Determination of the unit water content

The actual method of determining the unit water content differed from the method outlined in point 9 of the above procedure. Point 9 of the procedure was the outcome of the failure of the actual method.

The actual method was to first mix the stone, sand and cement. The cement content was 10 lbs. and the proportions were those found in point 8. To this mixture 8 lbs. of water was added and the batch was mixed for three minutes before a slump test was made. The slump was 1", the rodding was difficult and the plasticity, cohesiveness and finishing properties were all poor. This batch was poured back into the mixer and one lb. of water was added. After mixing for another three minutes a slump test was made. The slump was 3", the rodding was easy and the plasticity, cohesiveness and finishing properties were good. It was concluded that a total water content of 9 lbs. was correct.

(10) Computation of batch quantities

The amount of water absorbed by the stone = 0.4%. For the mix under consideration which had 10 lbs. of cement, the amount of water absorbed by the stone = 0.004x39.6 = 0.12 lbs. By the sand the amount = 0.002x36.4 = 0.08 lbs. Therefore the net free water in the mix = 9.0-0.12-0.08 = 8.8 lbs. The absolute volume of the batch = 8.8/62.5 + 36.4/2.70x62.5 + 39.6/2.70x62.5 + 10/3.15x62.5 = 0.642 cu.ft. Therefore the corrected total water content per cu.yd. = 9.0x27/0.642 = 378 lbs. Recipe 3, which provided concrete of suitable consistency, had a total water content of 353 lbs./cu.yd. It was decided to use a total water content of 360 lbs./cu.yd. The free water content was found by considering the absorption of the aggregates. The amount of water absorbed by the sand = 0.39x1580/100 = 6.15 lbs. The amount of water absorbed by the stone = 0.2x1715/100 = 3.4 lbs. making the total absorption equal to 9.5 lbs. The free water content = 360-9 = 351 lbs. per cu.yd.

The last step in the computation of the batch quantities was to calculate the mix quantities for a net water content of 351 lbs./cu.yd., a w/c ratio of 0.76 and a sand percentage of 48. The cement content = 351/0.76 =

462 lbs./cu.yd. The absolute volume of water + cement = 351/62.5 + 462/3.15x 62.5 = 7.95 cu.ft. The absolute volume of total aggregate = 27.0-7.95 = 19.05. The absolute volume of the sand = 0.48x19.05 = 9.15. The absolute volume of the stone = 19.05-9.15 = 9.90. The weight of sand = 2.70x62.5x1.002x9.15 = 1545 lbs./cu.yd. The weight of the stone = 2.70x62.5x9.9 = 1670 lbs./cu.yd. The mix proportions = 462/462: 1545/462: 1670/4.62 = 1: 3.34: 3.62.

(11) Calculation of particle interference

In the following table, d_o is defined as the volumetric density of any size group in a rodded condition and d_a is defined as the relative volumetric density of a particular size group in a concrete mix (10-16).

Particle Size	3/4	3/8	1/4	8	14	28	48	100	200
Wt.Ret. gms.	1037	1225	376	138	185	322	142	44	0
Abs.Vol. of Agg. c.c.	384	454	139	51	68	119	53	16	
Abs.Vol. Container	688	888	229	82	108	198	82	22	0
d_o	0.56	0.51	0.61	0.62	0.63	0.60	0.65	0.73	
Abs.Vol. in Mix	4.06	4.80	2.31	1.25	1.67	2.91	1.28	0.4	
d_a	0.15	0.21	0.127	0.08	0.11	0.22	0.13	0.05	
d_a/d_o	0.27	0.41	0.21	0.13	0.17	0.36	0.20	0.06	

Particle interference occurs where the value of d_a/d_o exceeds 0.3. Particle interference occurs in particle sizes 3/8 and 28. It is to be noted that for these particle sizes, the grading does not conform to the proposed grading bands of the A.C.I.(6-107). The particle interference of the 3/8 material could cause a strength reduction of 200 to 300 p.s.i. (10-16).

(12) Optimum sand percentage

As a reduction in sand content would increase particle interference, the sand percentage was not decreased. A sand percentage of 48 was chosen.

(13) Job curve

Introduction

The first three trial mixes which were computed by the method outlined in point 13 of the procedure were not successful. Their failure was because of the incorrect water content as found in point 9 of this section. The outcome of the first three mixes was the correct water content rather than the job curve. The job curve was determined after mixes 4 and 5 had been made.

Mixes 1, 2 and 3.

The standard w/c ratio was taken as 0.76 by weight. It was considered that this would produce 3500 lb. concrete. The w/c ratio range was 0.2. Therefore mix 1 had a w/c ratio of 0.66, mix 2 of 0.76 and mix 3 of 0.86. The unit water content was 360 lbs./cu.yd., nine pounds of which was considered to be absorbed by the aggregate, leaving a free water content of 351 lbs. per cu.yd. The sand percentage of mix 2, the standard mix, was taken as 48%.

Mix 2: Point 10 of this section contains the quantities for this mix. A volume of 1.2 cu.ft. was required for the 4 cylinders. The quantities for this volume are: water = 16 lbs., cement = 20.5 lbs., sand = 68.5 lbs., and stone = 74 lbs.

Mix 1: The cement content = the free water content divided by the w/c ratio = 351/0.66 = 531 lbs./cu.yd. The increase in the cement content of mix 1 over that of the standard mix 2 = 531-462 = 69 lbs. To keep the sum of the absolute volumes of the cement and sand of mix 1 equal to the corresponding sum of mix 2, the sand content of mix 1 must be decreased by the increase in the cement content multiplied by the specific gravity of the sand and divided by the specific gravity of the cement. The decrease in sand content for mix 1 = 69x2.7/3.15 = 59.2 lbs. Therefore the sand content for mix 1 = 1545/1.002 - 59.2 = 1485 lbs., where 1.002 is a factor allowing for the moisture content of the sand. The absolute volume of the stone is found by subtracting the sum of the absolute volumes of the water, cement and sand from 27 cu.ft. The sum of the absolute volumes of water, cement and sand = 351/62.5 + 531/3.15x62.5 + 1485/2.70x62.5 = 17.1 cu.ft. The absolute volume of the stone = 27.0-17.1 = 9.9 cu.ft. The weight of the stone = 9.9x2.7x62.5 = 1660 lbs./cu.yd. The proportions for mix 1 for the cement, sand and stone are: 531/531: 1485x1.002/531: 1660/531 = 1: 2.8: 3.13. The mix size was 1.2 cu.ft., which was sufficient to make 4 cylinders. The cement quantity for the mix = 1.2x531/27 = 23.6 lbs. The total water content = 0.66x23.6x360/351 = 16 lbs. The sand content = 2.8x23.6 = 66 lbs. and the stone content = 3.13x23.6 = 74 lbs. Mix 3: This mix was computed in the same manner as mix 1. The cement content = 18.1 lbs., the water content = 16 lbs., the sand content = 70.5 lbs., and the stone content = 74 lbs.

In mixing these three mixes the standard mix 2 was mixed first. The water content of 16 lbs. gave a very wet mix and a slump of 6". Mix 1 was made next but only 14 lbs. of water were added. This gave a slump of 5" and an excellent mix. In the case of mix 3 only 13 lbs. of water were added which resulted in a 3" slump and an excellent mix.

These changes in water content, which vitiated these mixes, would not have been necessary if the water content found in point 9 of this section had been correct. From the result of these mixes, the correct water content was 13 lbs. The corresponding water content per cu.yd. was next calculated for mix 3.

The sum of the absolute volumes of water, cement, sand and stone =

13/1.025x62.5 + 18.1/3.15x62.5 + 70.5/2.7x62.5 + 74/2.70x62.5 = 1.15 cu.ft. Therefore the total water content per cu.yd. = 13x27/1.15 = 304 lbs. The cement content/cu.yd. for mix 3 was 18.1x27/1.15 = 423 lbs. The sand content was 70.5x27/1.15 = 1650 lbs. and the stone content was 74x27/1.15 = 1735 lbs. This gave a sand percentage of mix 3 of 48 and a w/c ratio of 0.71.

The 4 cylinders, 6" diam. by 12" high, made from mix 3 tested at 96.4, 82.9, 91.3 and 87.1 kips. This gave a unit compressive stress of 3,150 lbs.

Mixes 4 and 5.

The standard for these mixes was mix 3. The range of the w/c ratios was from 0.60 to 0.85. As in the case of mix 3, the total water content for mixes 4 and 5 was 304 lbs./cu.yd. Considering the aggregate to absorb 9 lbs., this gave a free water content of 295 lbs./cu.yd. The sand percentage of mix 3, the standard, was 48. The size of mix for mixes 4 and 5 was 0.6 cu.ft. which was sufficient to provide two cylinders/mix. A larger mix, sufficient to provide four cylinders, would have been used if the cement supply had been sufficient.

Mix 4: With a water cement ratio of 0.85 and a free water content of 295 lbs./cu.yd., the cement content = 295/0.85 = 348 lbs. Considering mix 3 as standard, the decrease in cement content = 423-348 = 75 lbs. The necessary increase in the sand content = 2.7x75/3.15 = 67 lbs. The sum of the absolute volumes of water, cement and sand = 295/62.5 + 348/3.15x62.5 + (1650+67)/2.7x62.5 = 16.7 cu.ft. The absolute volume of the stone = 27.0-16.7 = 10.3 cu.ft. The weight of the sand/cu.yd.= (1650+67)x1.002 = 1720 lbs. The weight of stone/cu.yd. = 2.7x62.5x10.3 = 1740 lbs. The proportions by weight of the cement, sand and stone were: 348/348: 1720/348: 1740: 348 = 1: 4.95: 5.00. For a batch of 0.6 cu.ft. the quantities were: cement = 348x0.6/27 = 7.73 lbs., water = 0.85x7.73x 1.025 = 6.74 lbs., sand = 4.95x7.73 = 38.2 lbs. and stone = 5.00x7.73 = 38.6 lbs.

Mix 5: The water cement ratio = 0.60, and the free water content = 295 lbs./cu.yd. Mix 3 was considered as standard. The design was similar to that of mix 4, the quantities for a batch of 0.6 cu.ft. were: cement = 10.9 lbs., water = 6.73 lbs., sand = 35.3 lbs. and stone = 38.8 lbs.

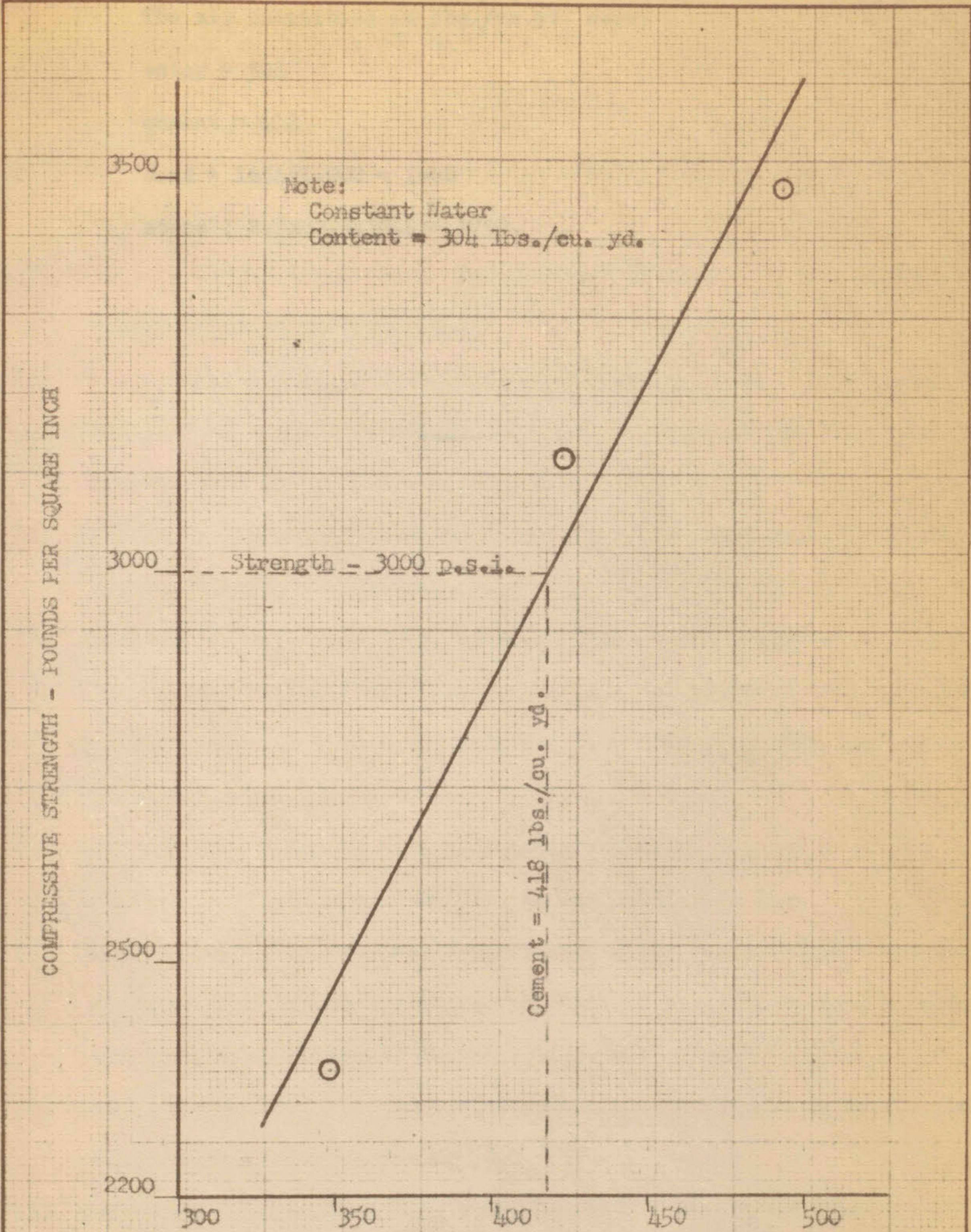
Results of mixes 3,4 and 5

The job curve was prepared from the results of mixes 3, 4 and 5. The relevant data are given in the following table.

Mix Number	3	4	5
w/c ratio	0.71	0.85	0.60
Cement, lbs./cu.yd.	423	348	492
Total water, lbs./cu.yd.	304	304	304
Sand percentage	48	49	47
Slump, inches	4	3	4
Rodability	Good	Good	Good
Finish	do.	Med.	do.
Segregation	do.	Good.	do.
Plasticity	do.	do.	do.
Cohesiveness	do.	do.	do.
Cylinder values, kips	96.4 82.9 91.3 87.1	68.7 64.8	95.8 102.0
Average strength, p.s.i.	3,150	2,360	3,490

RESULTS

For 3,000 lb. concrete, the job curve gives a cement content of 418 lbs./cu.yd. and a total water content of 304 lbs. per cu.yd. Considering mix 3 as standard, the decrease in cement = 423-418 = 5 lbs. The corresponding increase in sand content = 2.7x5/3.15 = 4.3 lbs. The sum of the absolute volumes of water, cement and sand was 295/62.5 + 418/3.15x62.5 + (1650+4.3)/2.7x62.5 = 16.6 cu.ft. The absolute volume of the stone = 27.0-16.6 = 10.4 cu.ft.



Note:
Constant Water
Content = 304 lbs./cu. yd.
3500
3000
2500
2200
Strength - 3000 p.s.i.
Cement = 418 lbs./cu. yd.
COMPRESSIVE STRENGTH - POUNDS PER SQUARE INCH
300
350
400
450
500
CEMENT CONTENT - POUNDS PER CUBIC YARD

JOB CURVE

The mix quantities in lbs./cu.yd. were:

water = 304

cement = 418

sand = 1654x1.002 = 1660

stone ; 2.7x62.5x10.4 = 1750.

APPENDIX

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(4) Canadian Engineering Standards Association, Standard Specification for Concrete and Reinforced Concrete, A-23, 1942.

(5) Canadian Engineering Standards Association, Standard Specifications for Portland Cement, A-57, 1940.

(6) American Concrete Institute, Proposed Recommended Practice for the Design of Concrete Mixes, Journal of the A.C.I., Vol.15, No.2, November 1943.

(7) American Concrete Institute, Progress with Concrete, 1923-1948, Journal of the A.C.I., Vol.19, No.8, April 1948.

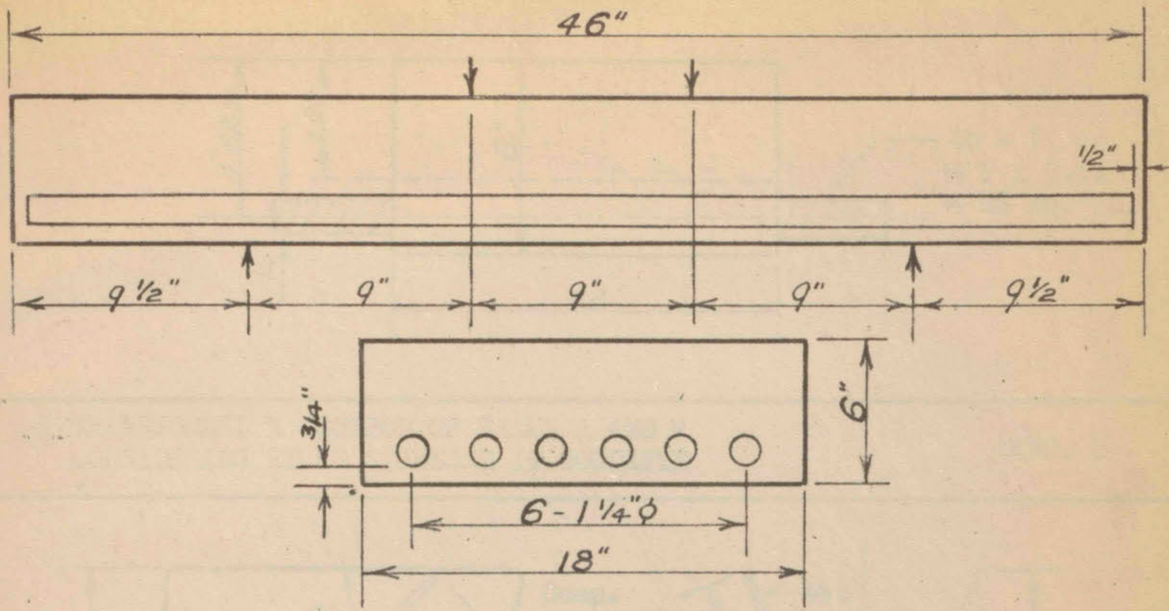
(8) Lash and Lee, Notes on Design of Concrete Mixes, Journal of the Engineering Institute of Canada, August 1943, page 444.

(9) Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete, August 1944.

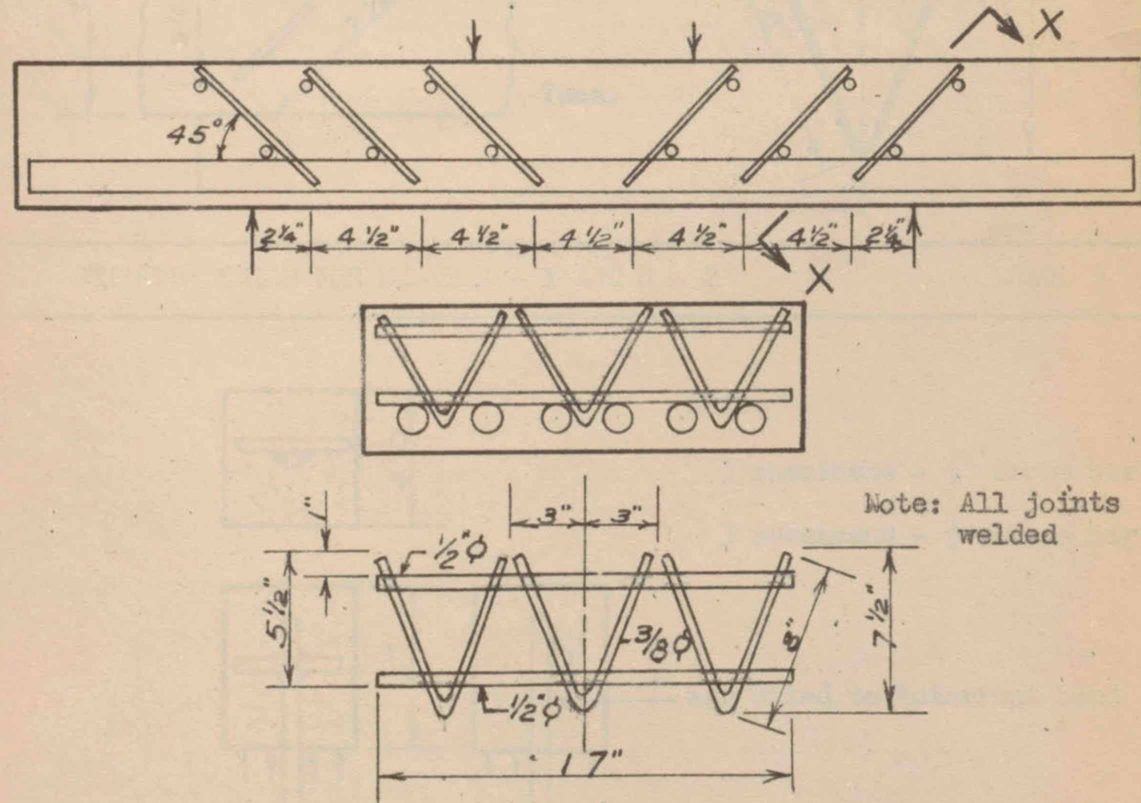
(10) McGill University, Strength of Materials Laboratory Course No.88, E.U.S. Publications Board, December 1948.

(11) Swayze and Gruenwald, Concrete Mix Design - a Modification of the Fineness Modulus Method, Journal of the American Concrete Institute, Volume 18, No.7, March 1947.

LIST OF ILLUSTRATIONS



BEAM A

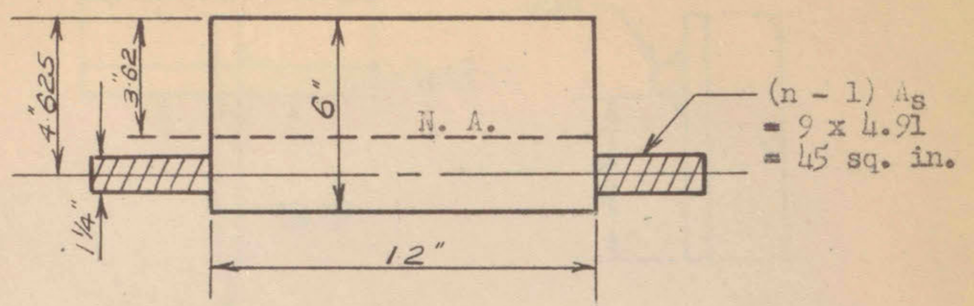


Section X - X

BEAM B

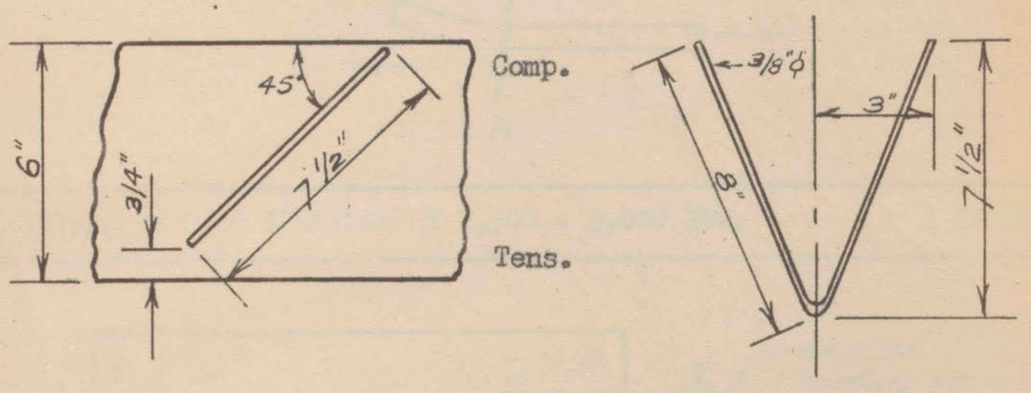
BEAMS A AND B

DRWG. 1



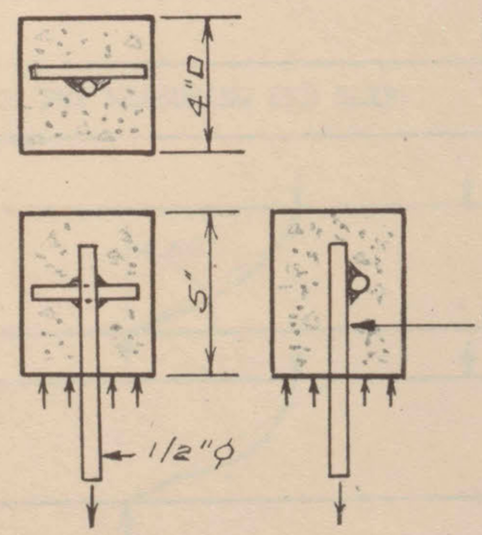
TRANSFORMED X SECTION OF BEAMS A AND B
CONSIDERING FIBRE STRESSES IN CONCRETE

DRWG. 2



STIRRUP DESIGN FOR BEAMS B - 1 AND B - 2

DRWG. 3



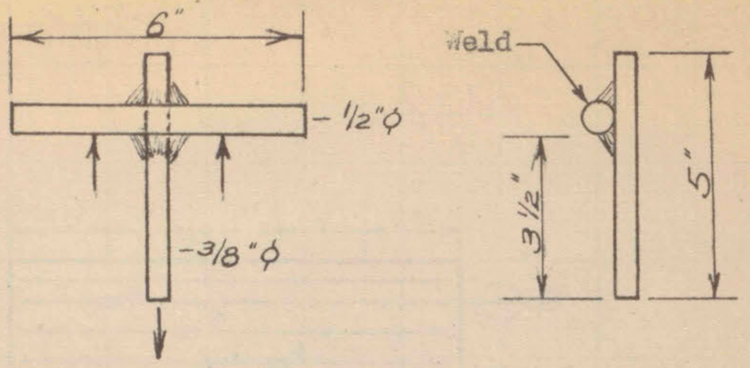
Note:

3 specimens - ¼" cross bar

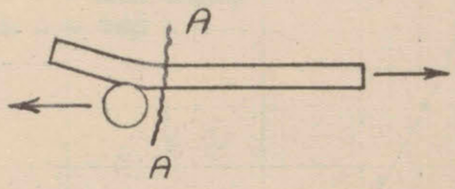
3 specimens - ½" cross bar

TEST SPECIMENS FOR BEARING STRENGTH OF CONCRETE

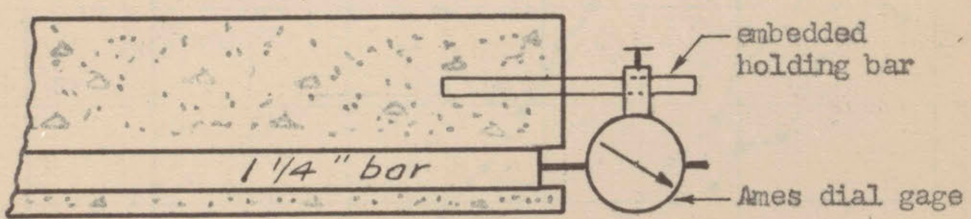
DRWG. 4



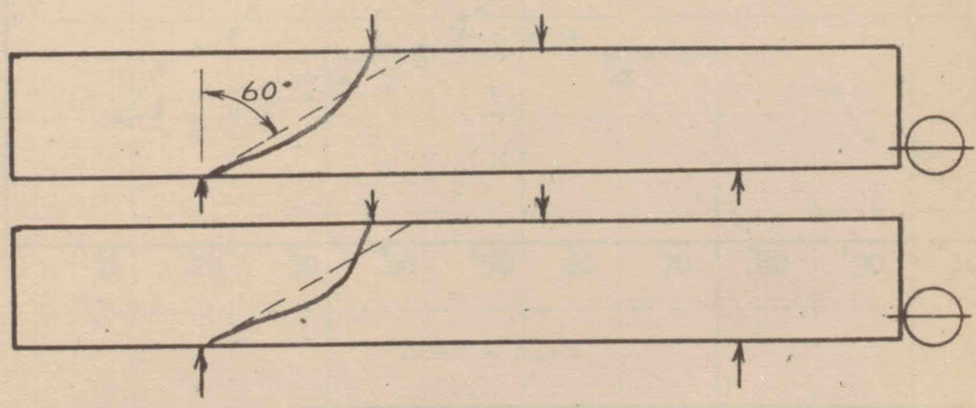
SPECIMENS TO DETERMINE WELD STRENGTH DRWG. 5



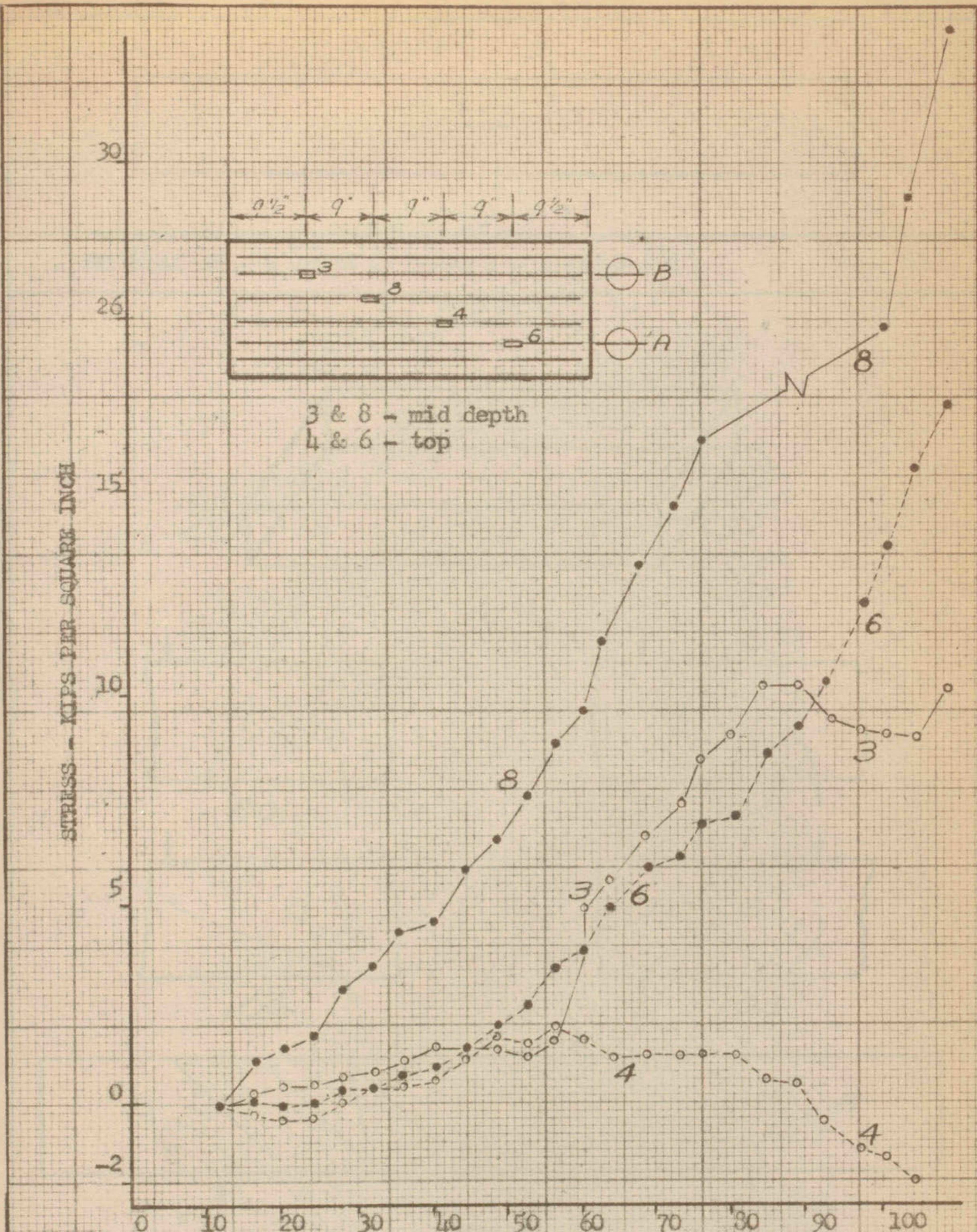
SHAPE OF WELD SPECIMEN AT 2,500 - 3,000 lbs. DRWG. 6



BOND GAGE FOR MEASURING END SLIP DRWG. 7

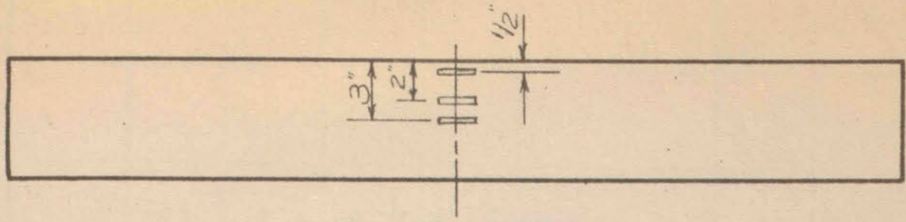


TEST PHENOMENA OF BEAM A - 1 DRWG. 8

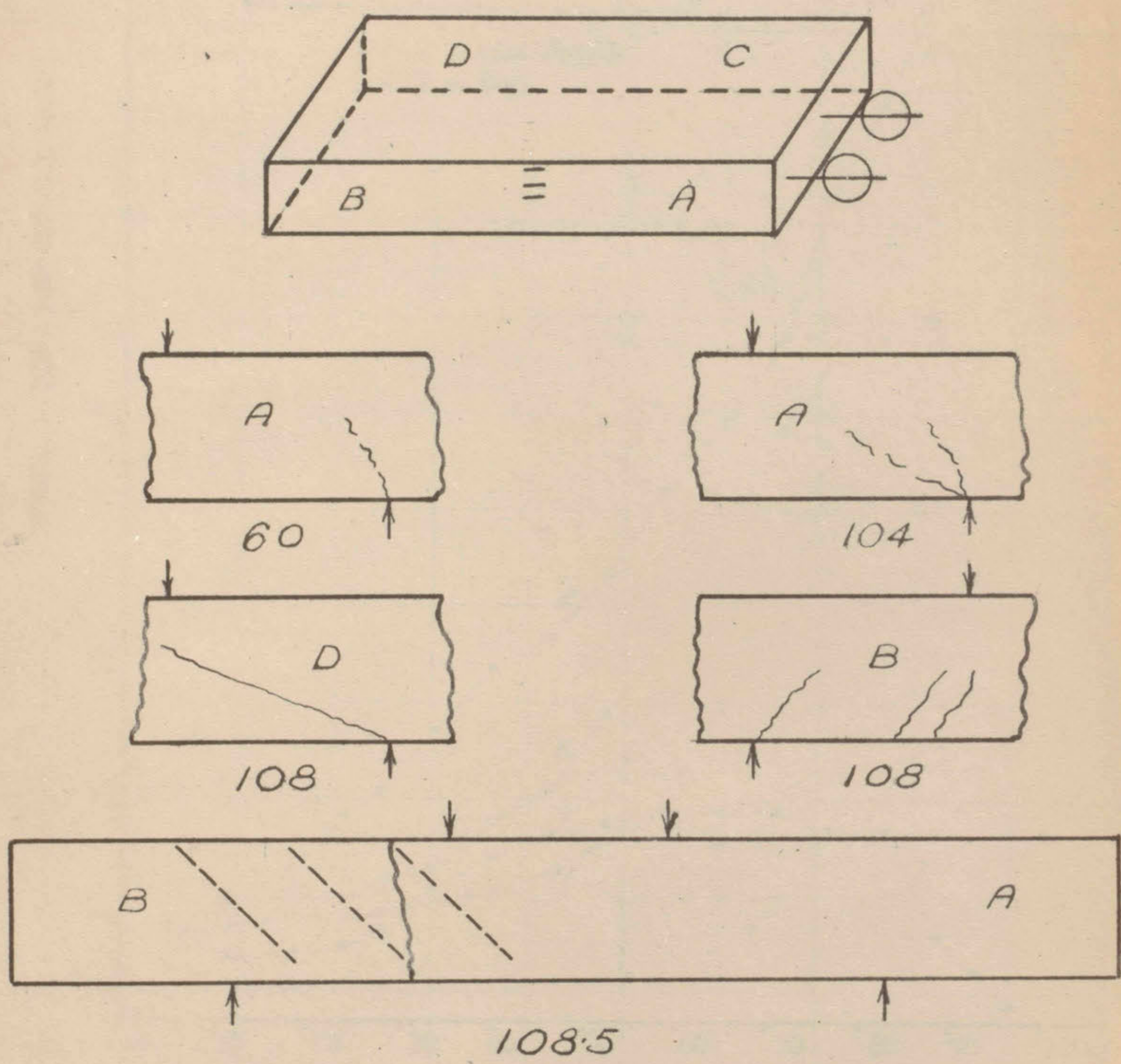


BEAM B - 1 TENSILE SR - 4 GAGES

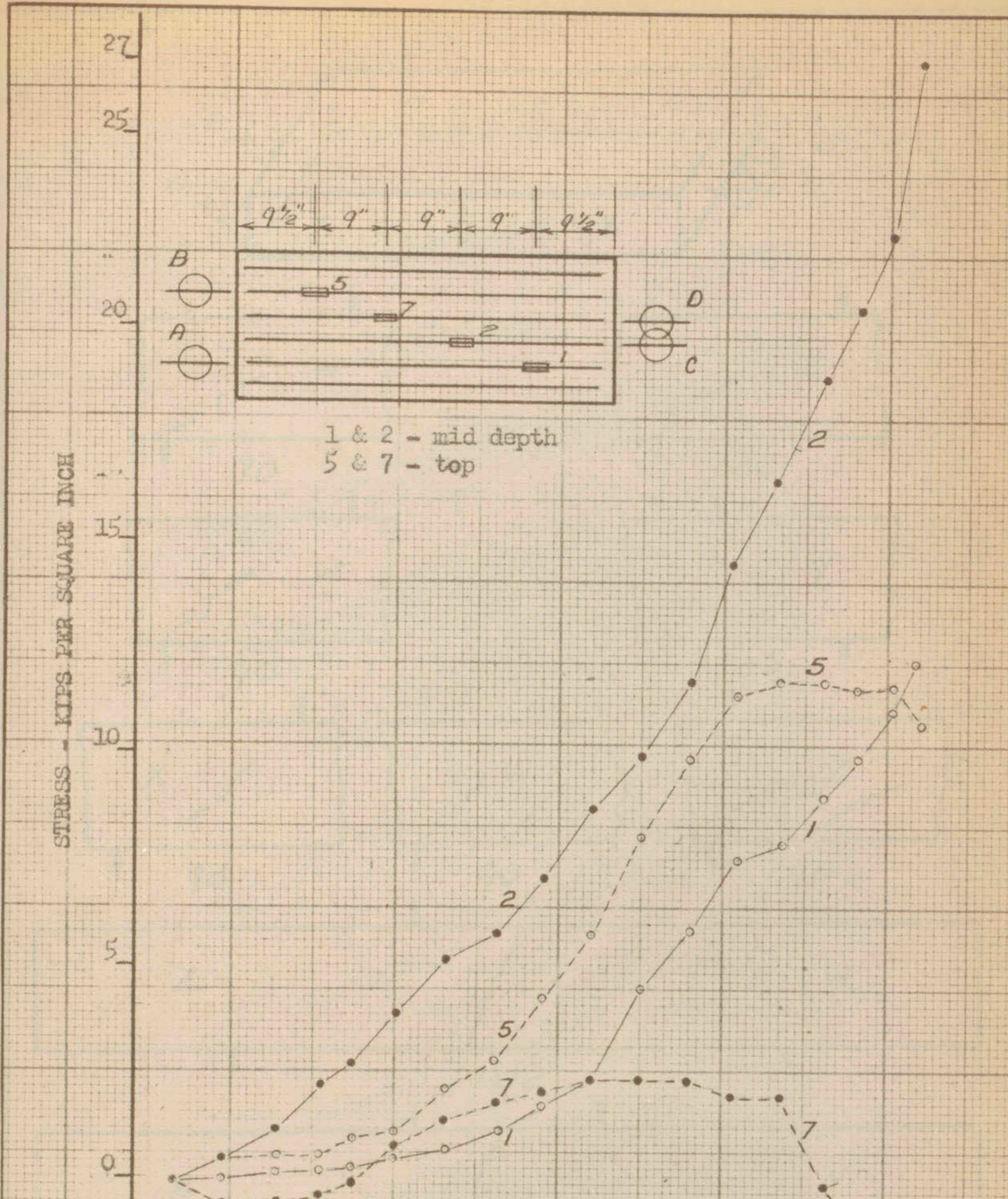
DR.G. 9



EXTERNAL SR - 4 GAGES ON BEAMS B - 1 AND B - 2 DRWG. 10

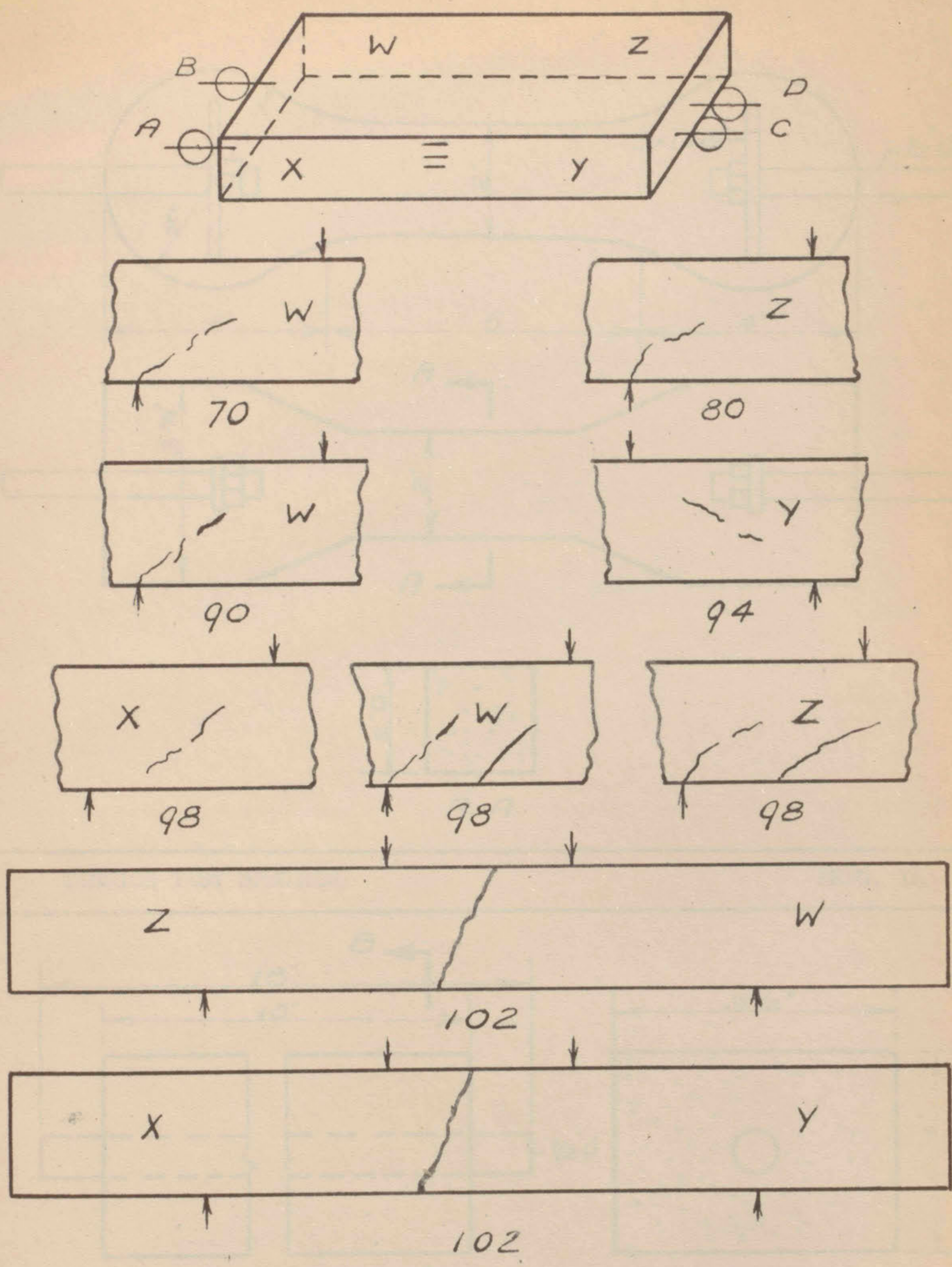


TEST PHENOMENA OF BEAM B - 1 DRWG. 11



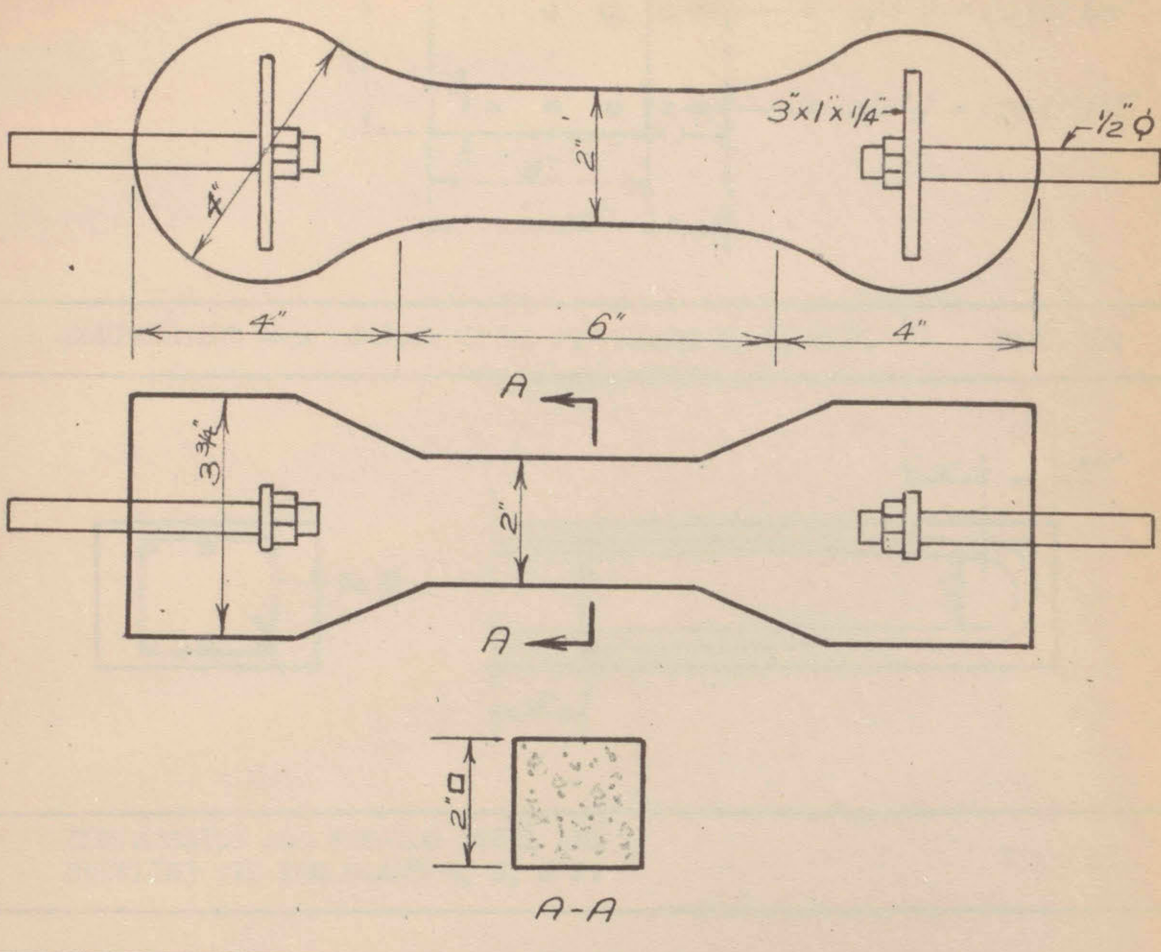
BEAM B - 2 TENSILE SR - 4 GAGES

DRWG. 12

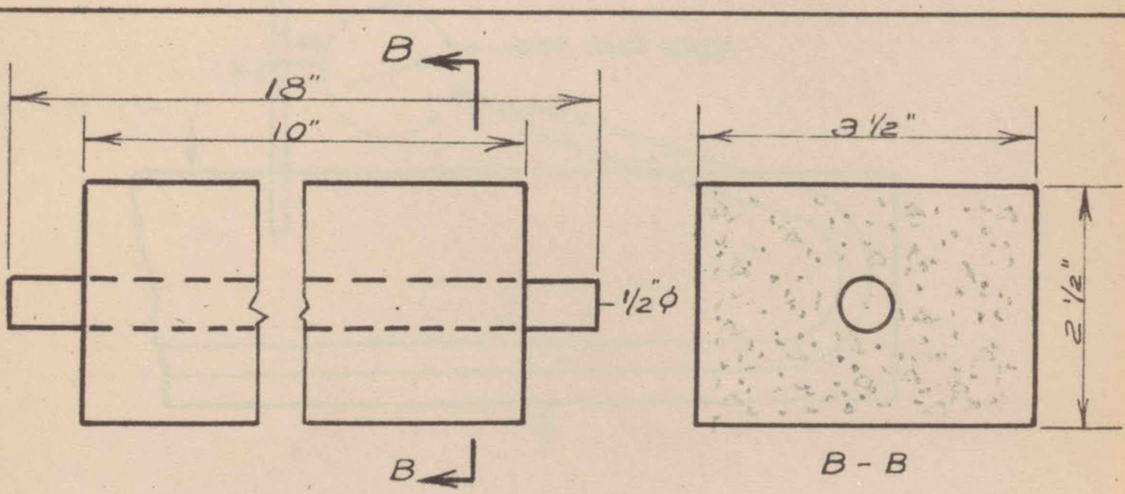


TEST PHENOMENA OF BEAM B - 2

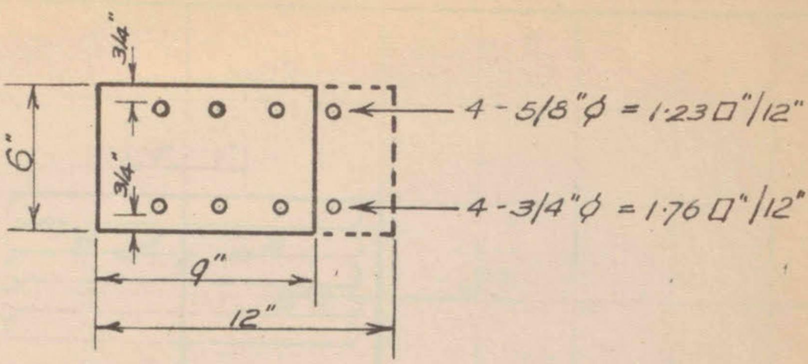
DR.WG. 13



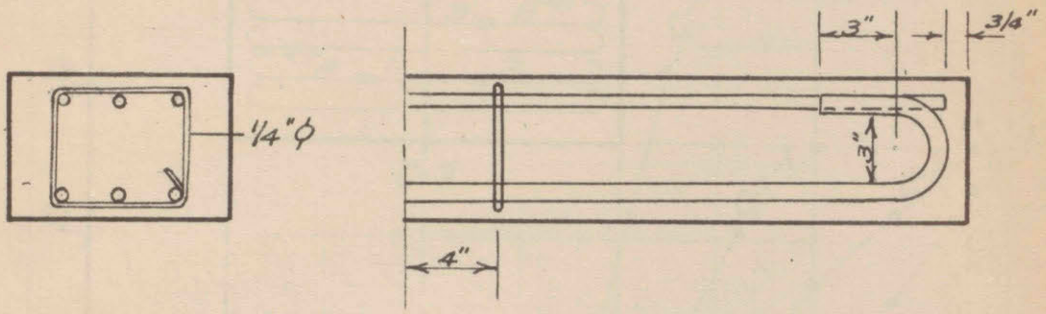
TENSION TEST SPECIMEN DRWG. 14



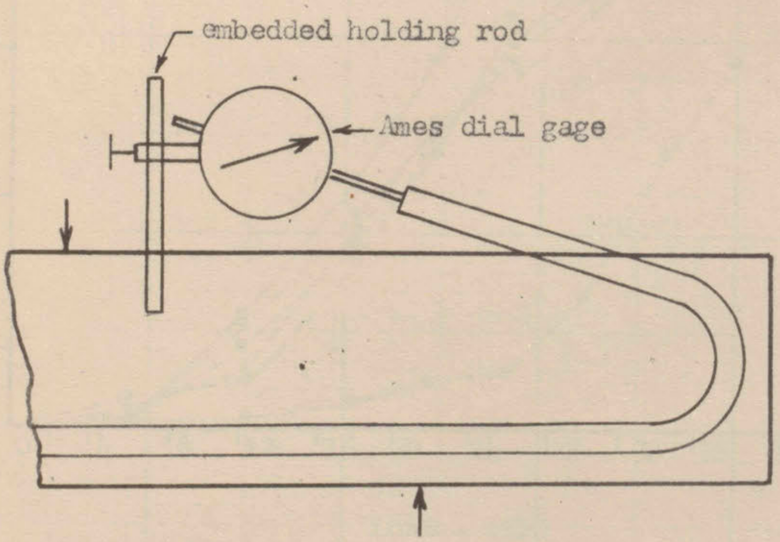
TEST SPECIMEN DRWG. 15



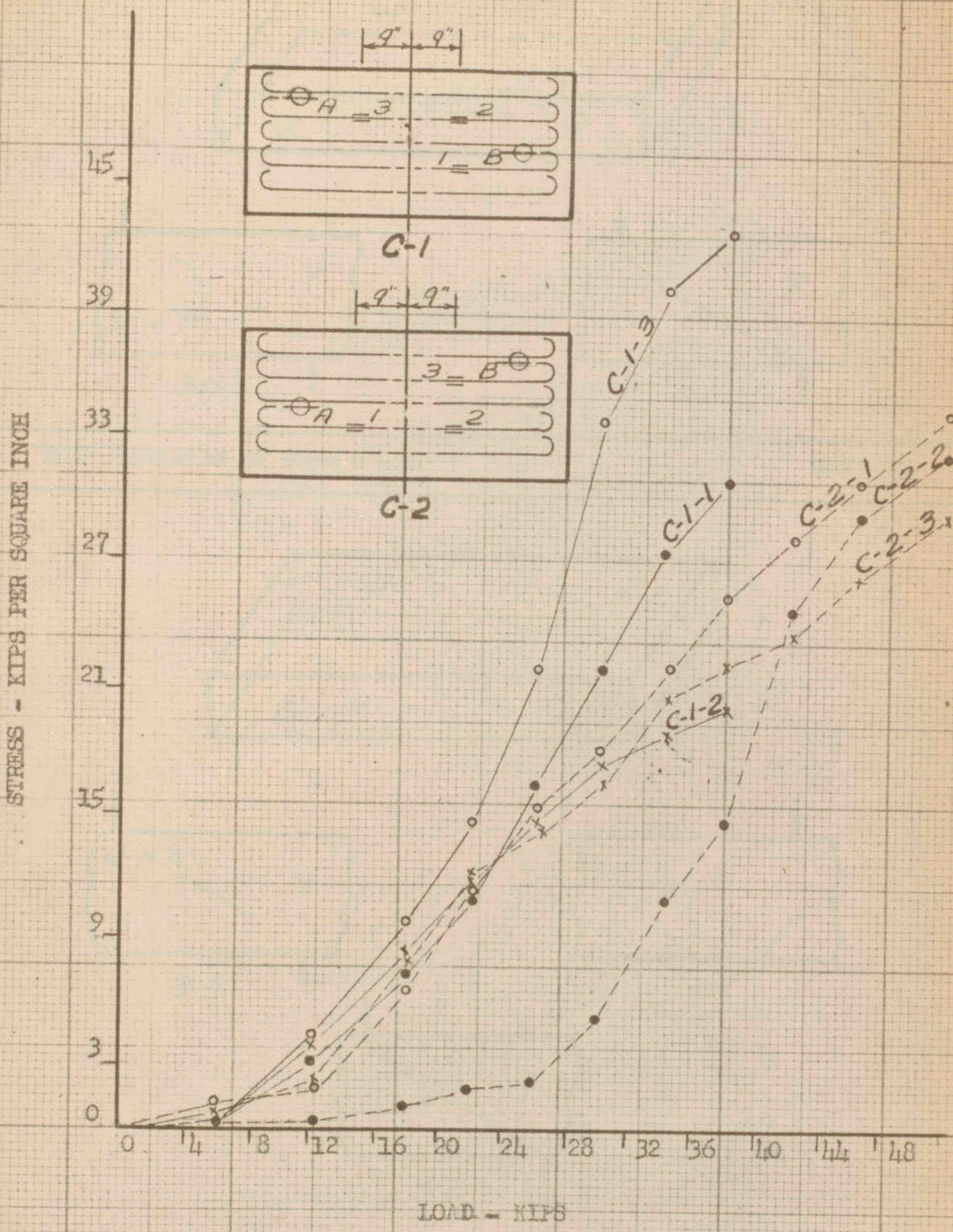
COMPRESSION AND TENSION STEEL FOR BEAMS D, E, & F. DRWG. 16



COMPRESSION AND TENSION STEEL AND
BUCKLING TIE FOR BEAMS D, E, & F. DRWG. 17

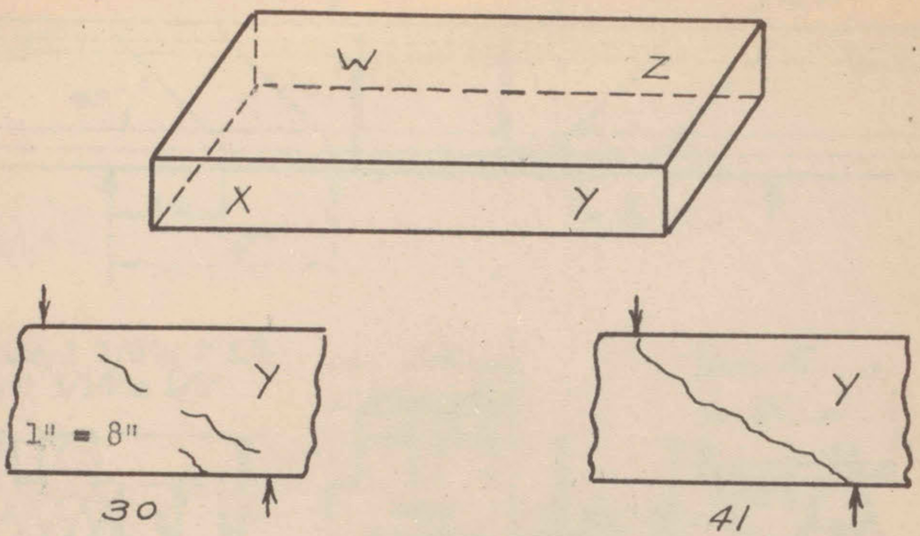


BOND GAGES FOR BEAMS C, D, E & F. DRWG. 18

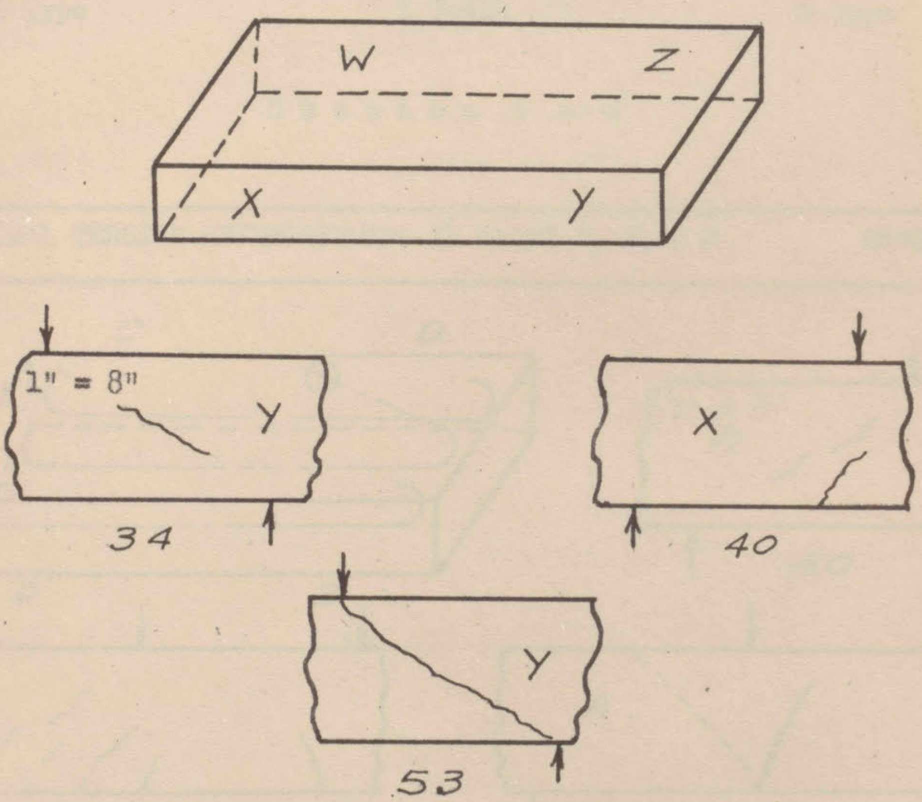


SR - 4 GAGES ON BEAMS C - 1 AND C - 2

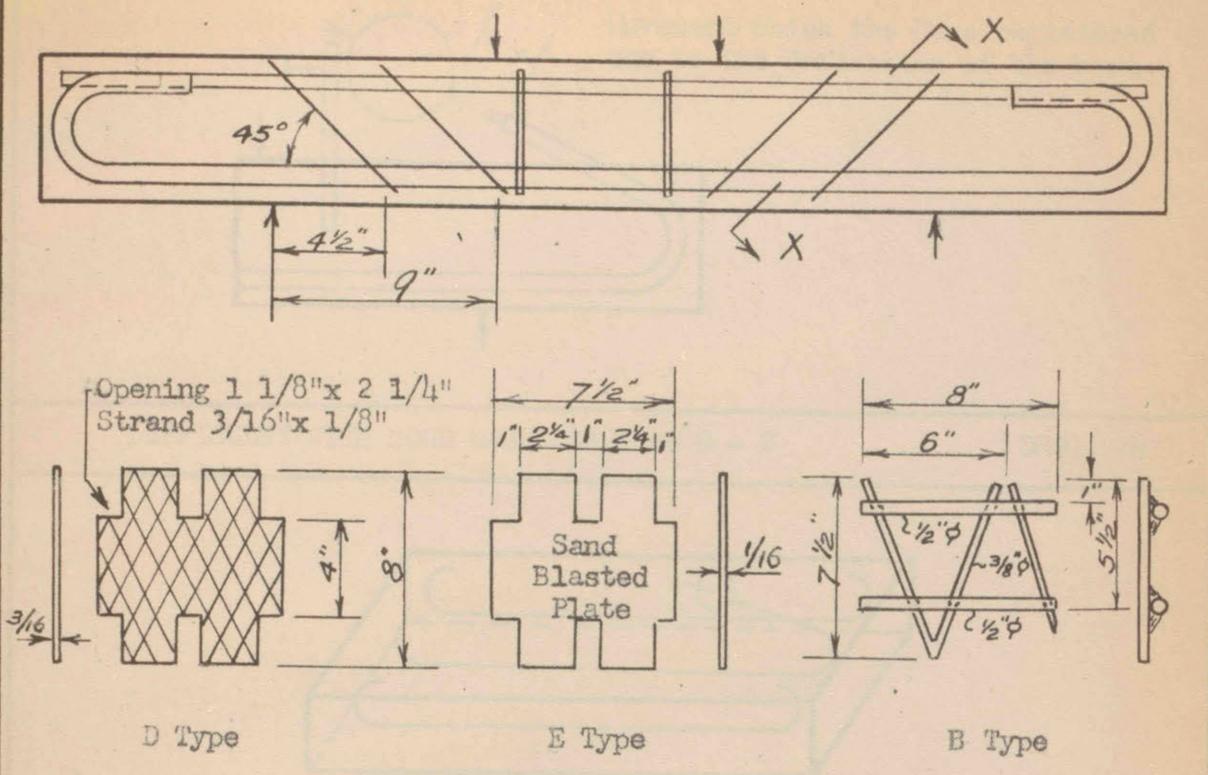
DRWG. 19



TEST PHENOMENA OF BEAM C - 1 DRWG. 20

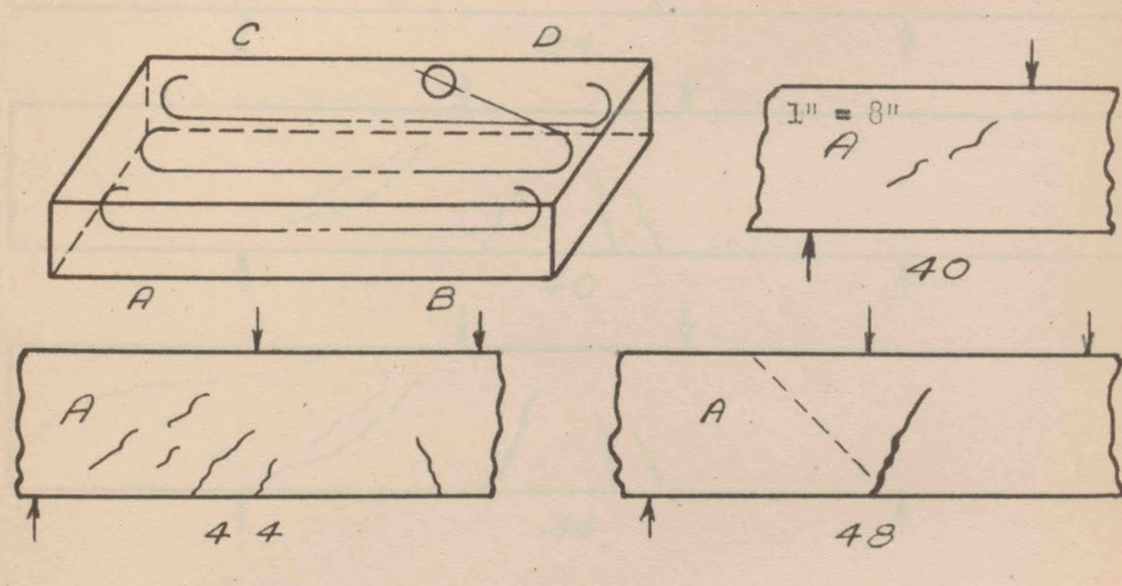


TEST PHENOMENA OF BEAM C - 2 DRWG. 21

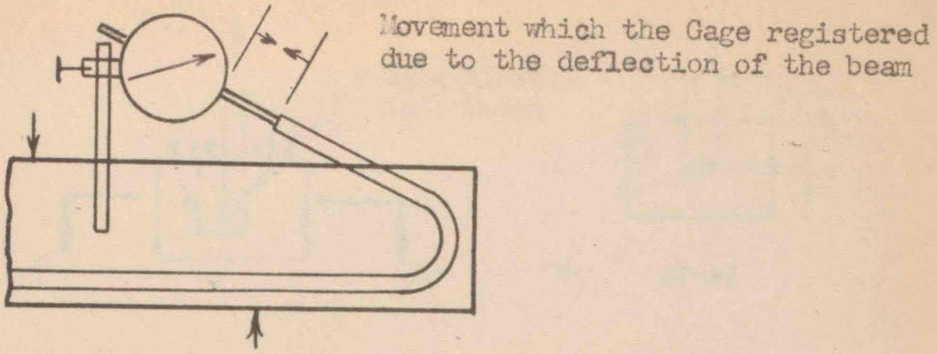


Section X - X

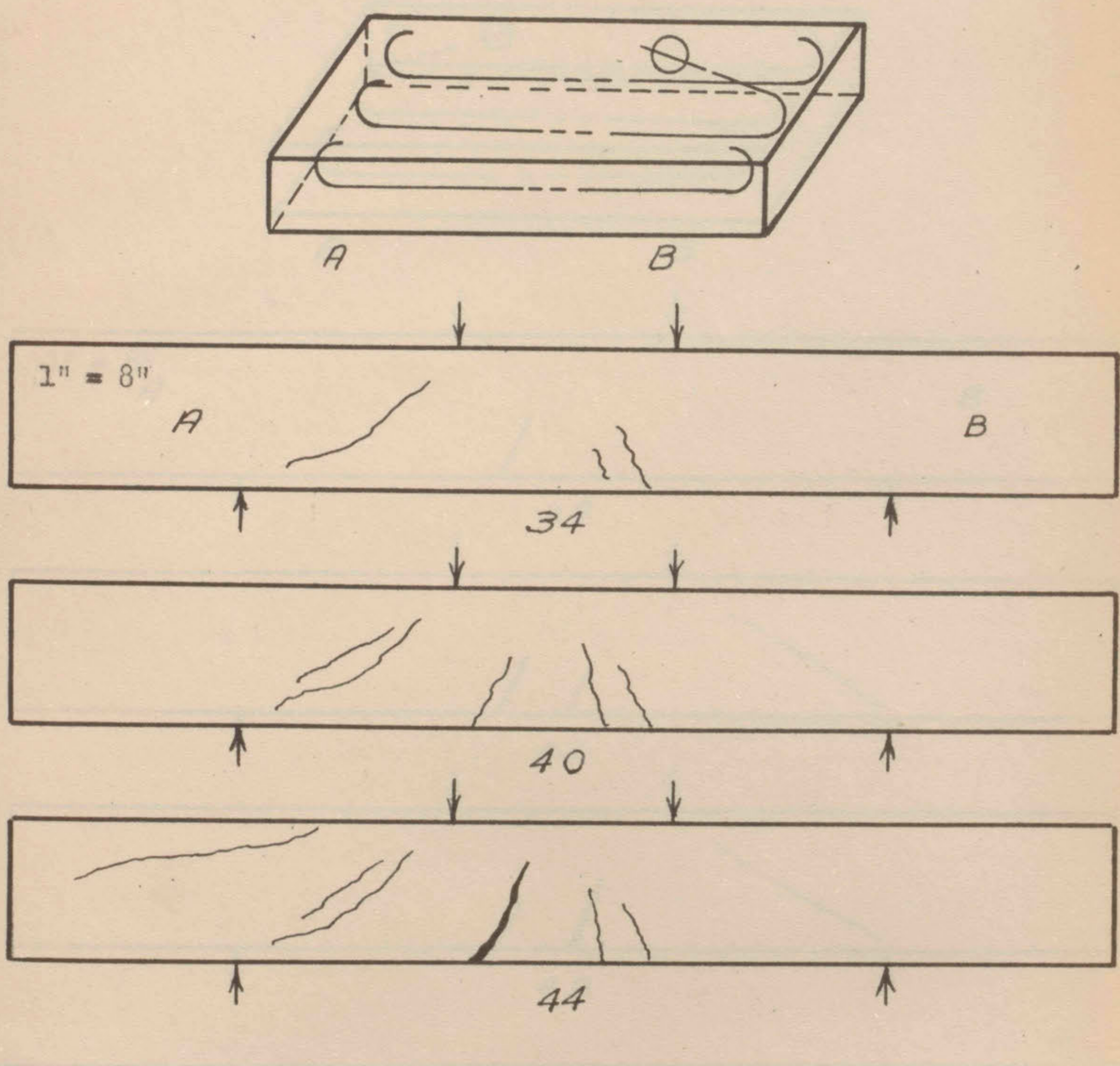
DIAGONAL TENSION REINFORSEMENT IN BEAMS D, E, & F. DRWG. 22



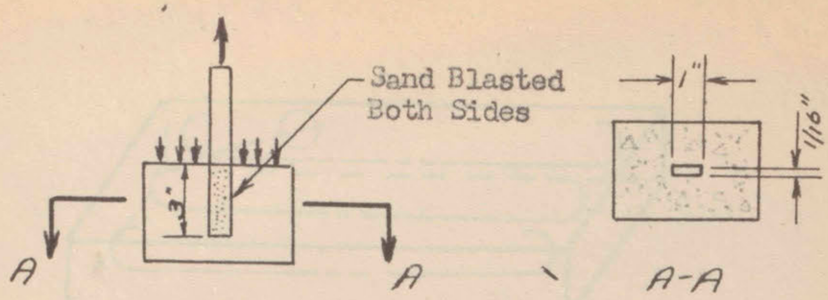
TEST PHENOMENA OF BEAM D - 1 DRWG. 23



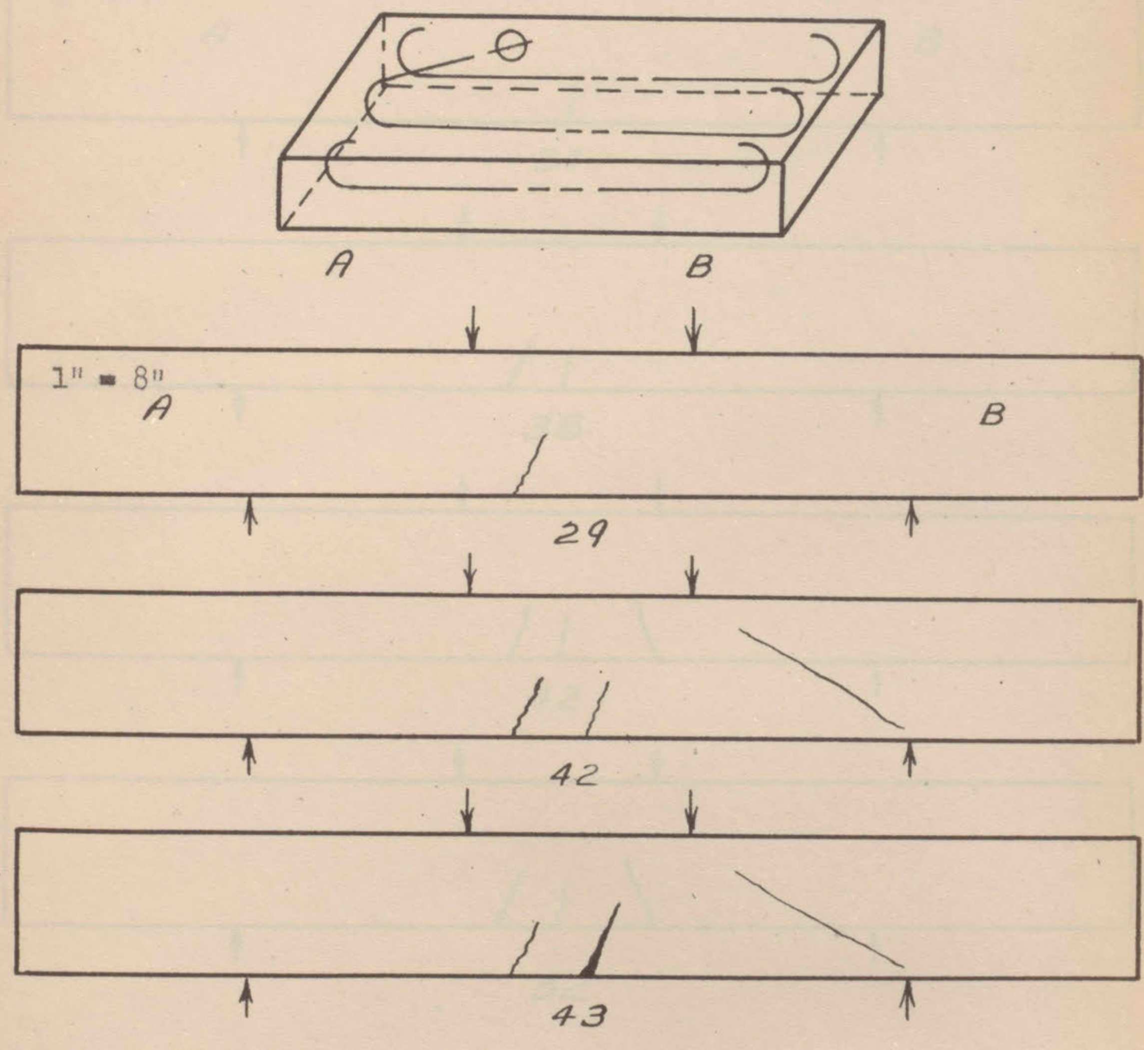
DIFFICULTY WITH BOND GAGE FOR BEAM D - 2 DRWG. 24



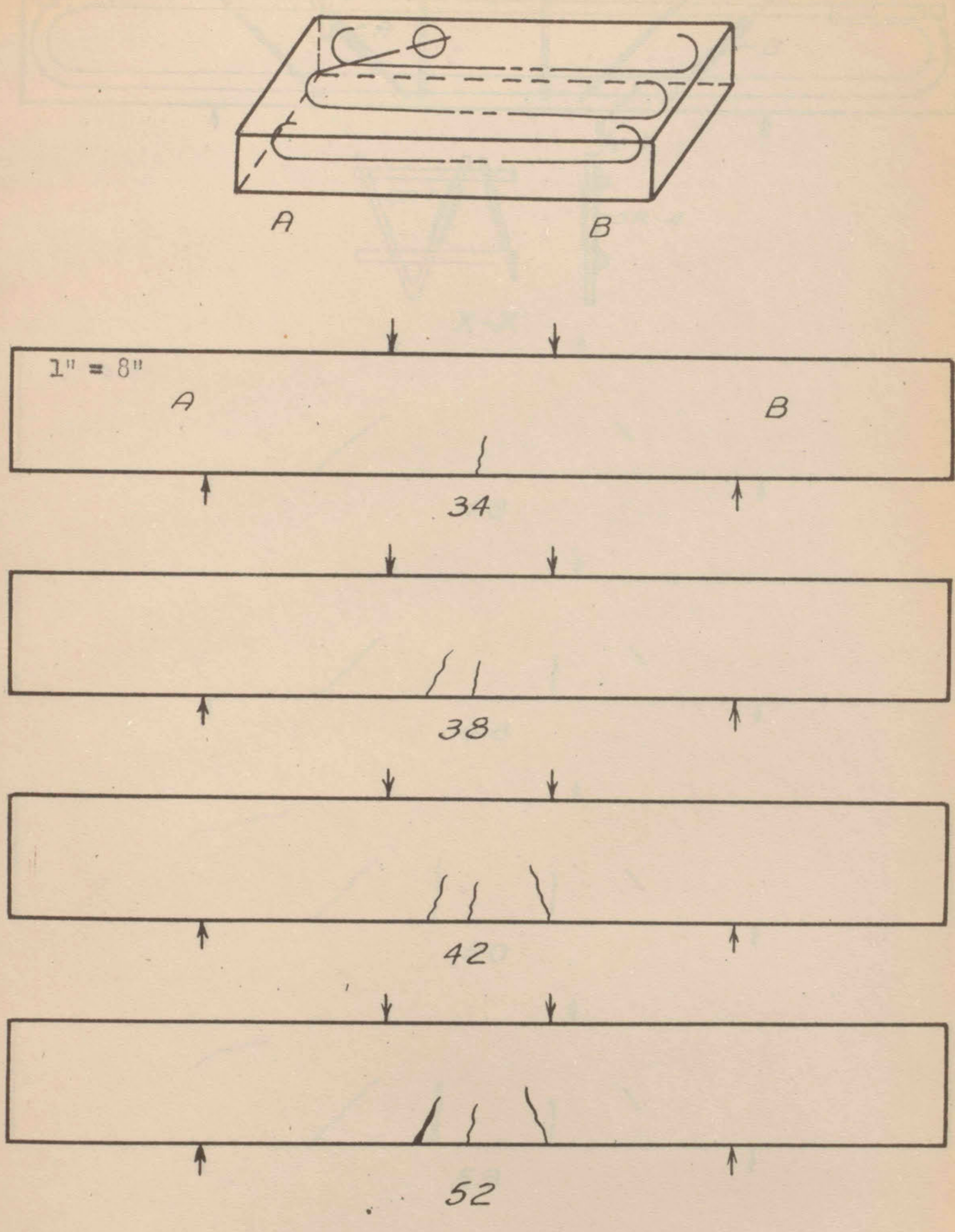
TEST PHENOMENA OF BEAM D - 2 DRWG. 25



BOND SPECIMENS FOR BEAMS E - 1 AND E - 2 DRWG. 26

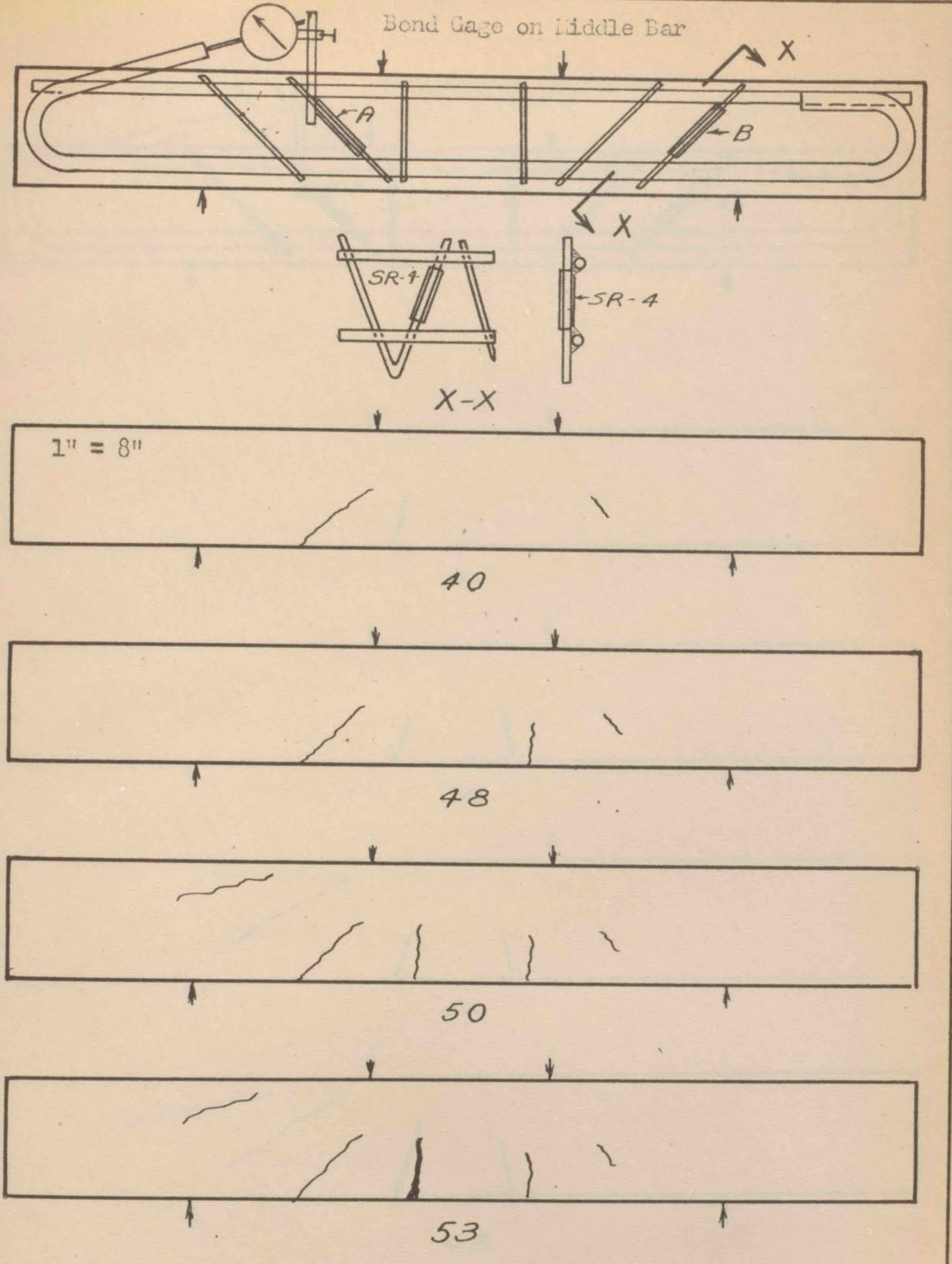


TEST PHENOMENA OF BEAM E - 1 DRWG. 27

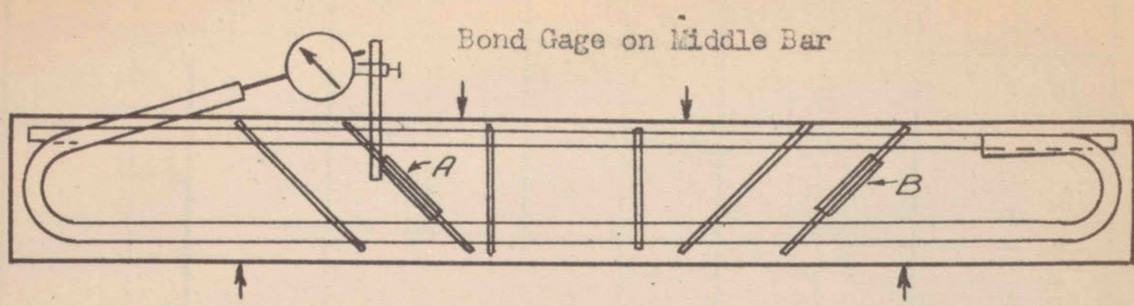


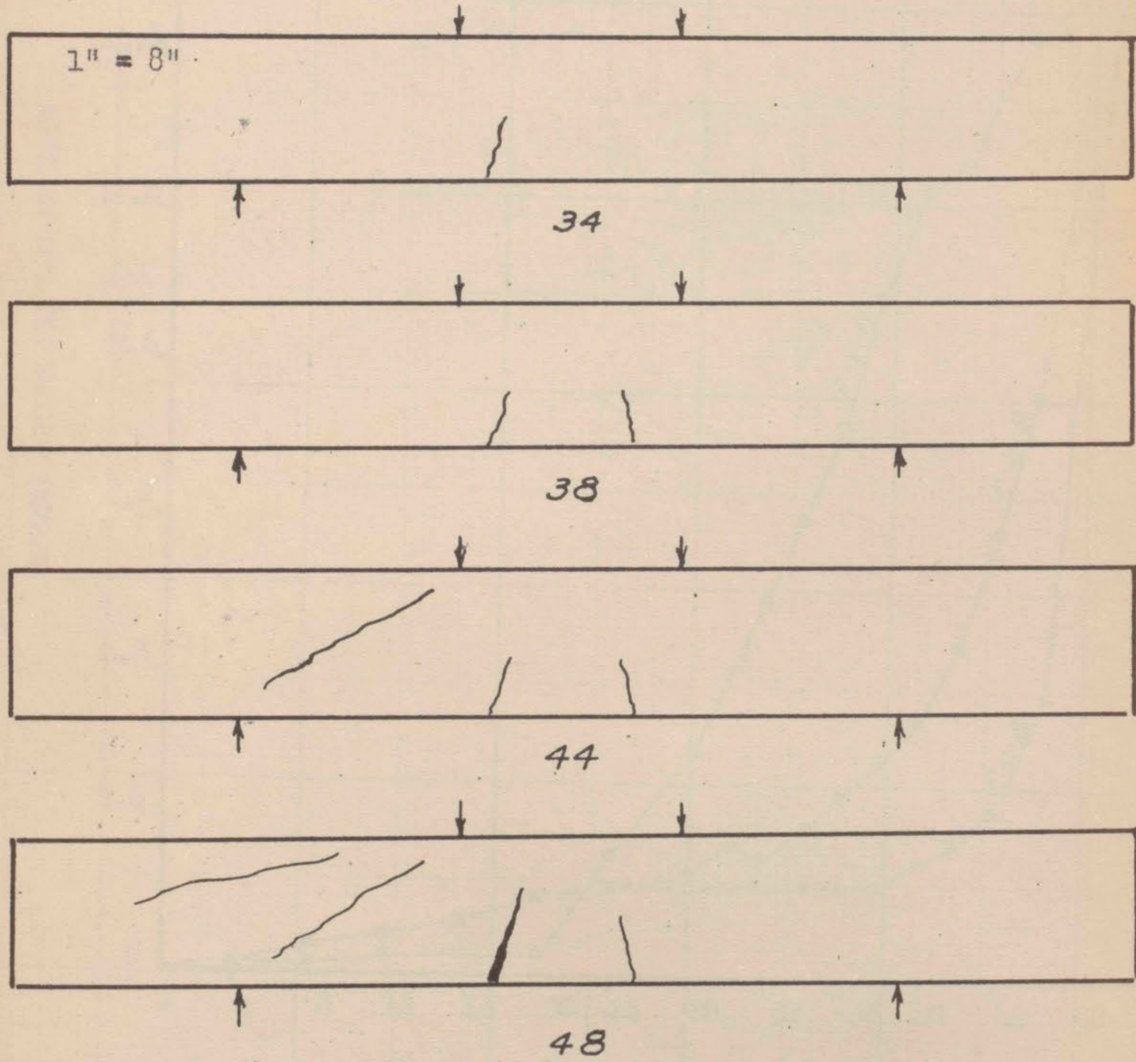
TEST PHENOMENA OF BEAM E - 2

DRWG. 28



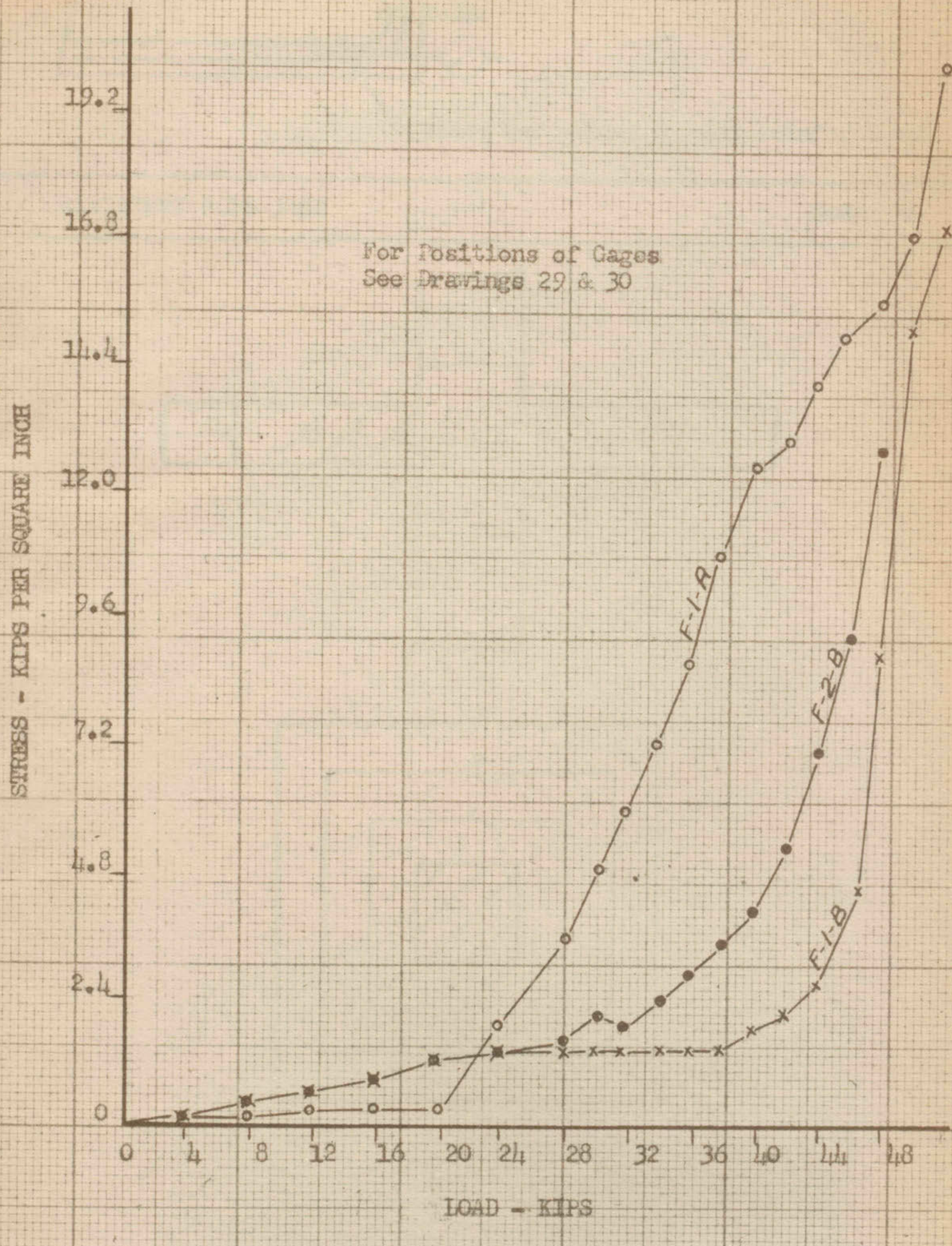
TEST PHENOMENA OF BEAM F - 1 DRWG. 29



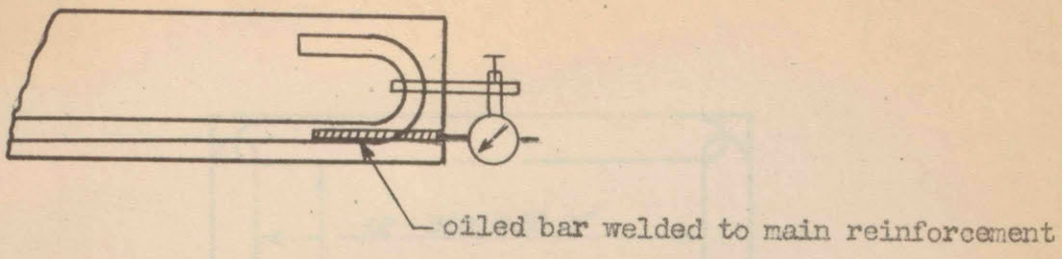


TEST PHENOMENA OF BEAM F - 2

DRWG. 30



STRESS IN STIRRUPS OF BEAMS F - 1 AND F - 2 DRWG. 31



RECOMMENDED BOND GAGE DRWG. 32

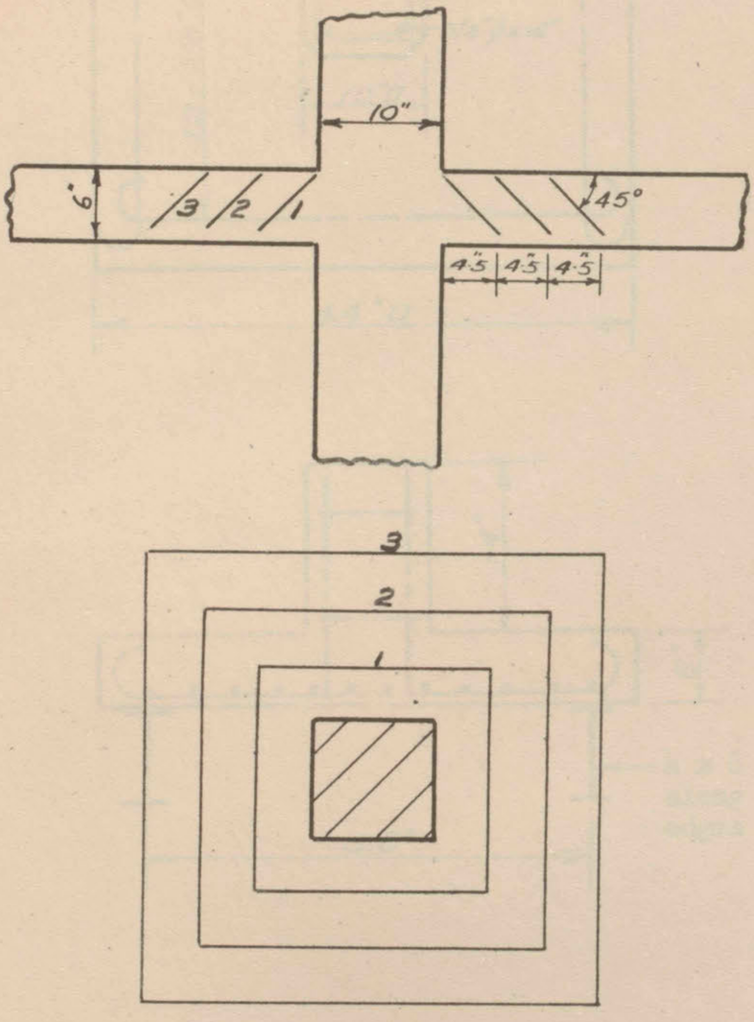
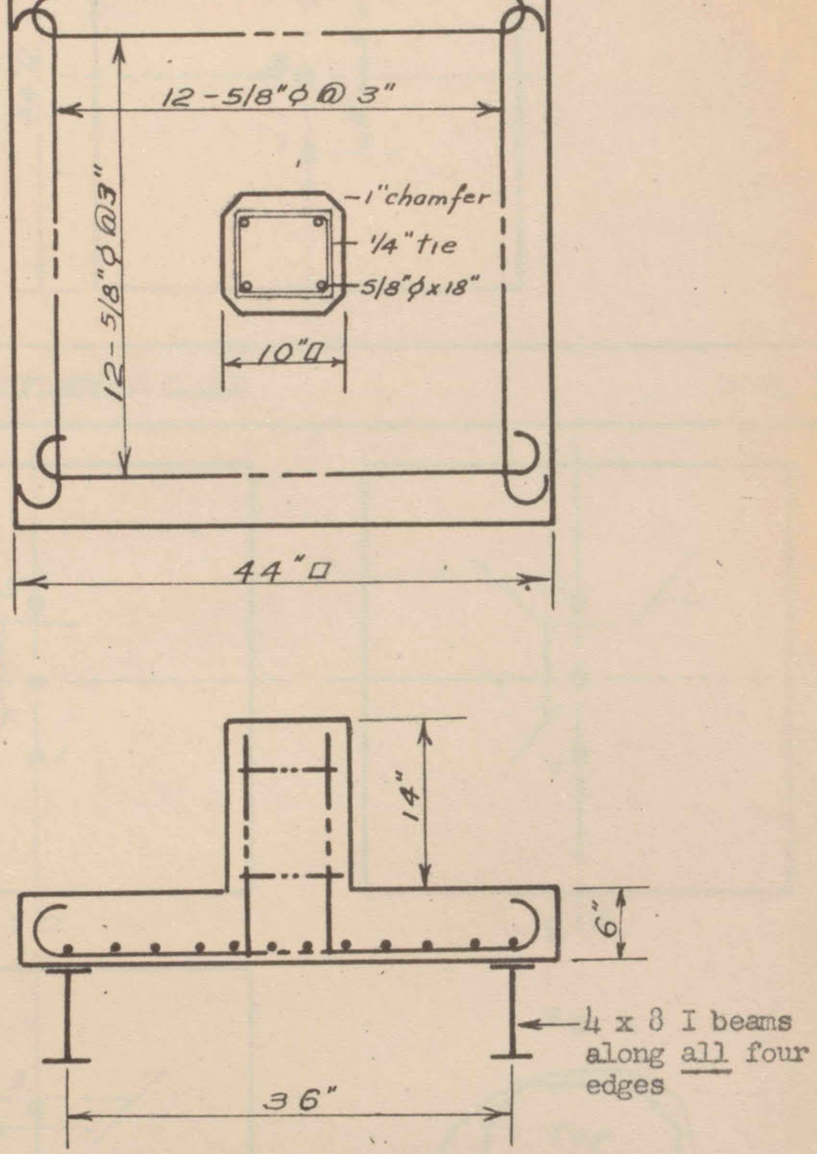
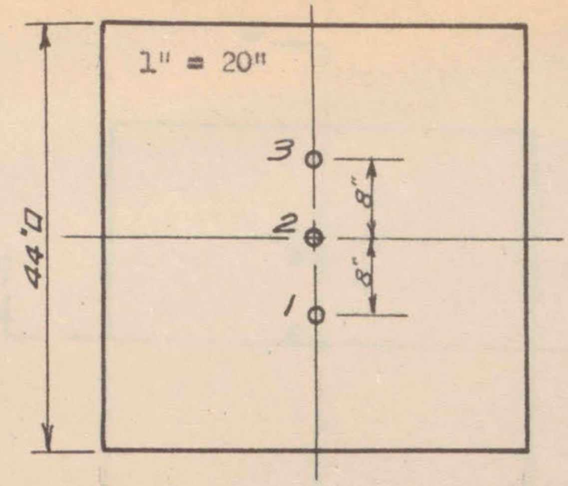


DIAGRAM FOR COST ESTIMATING THE DIAGONAL TENSION REINFORCEMENT IN A FLATE PLATE BUILDING DRWG. 33

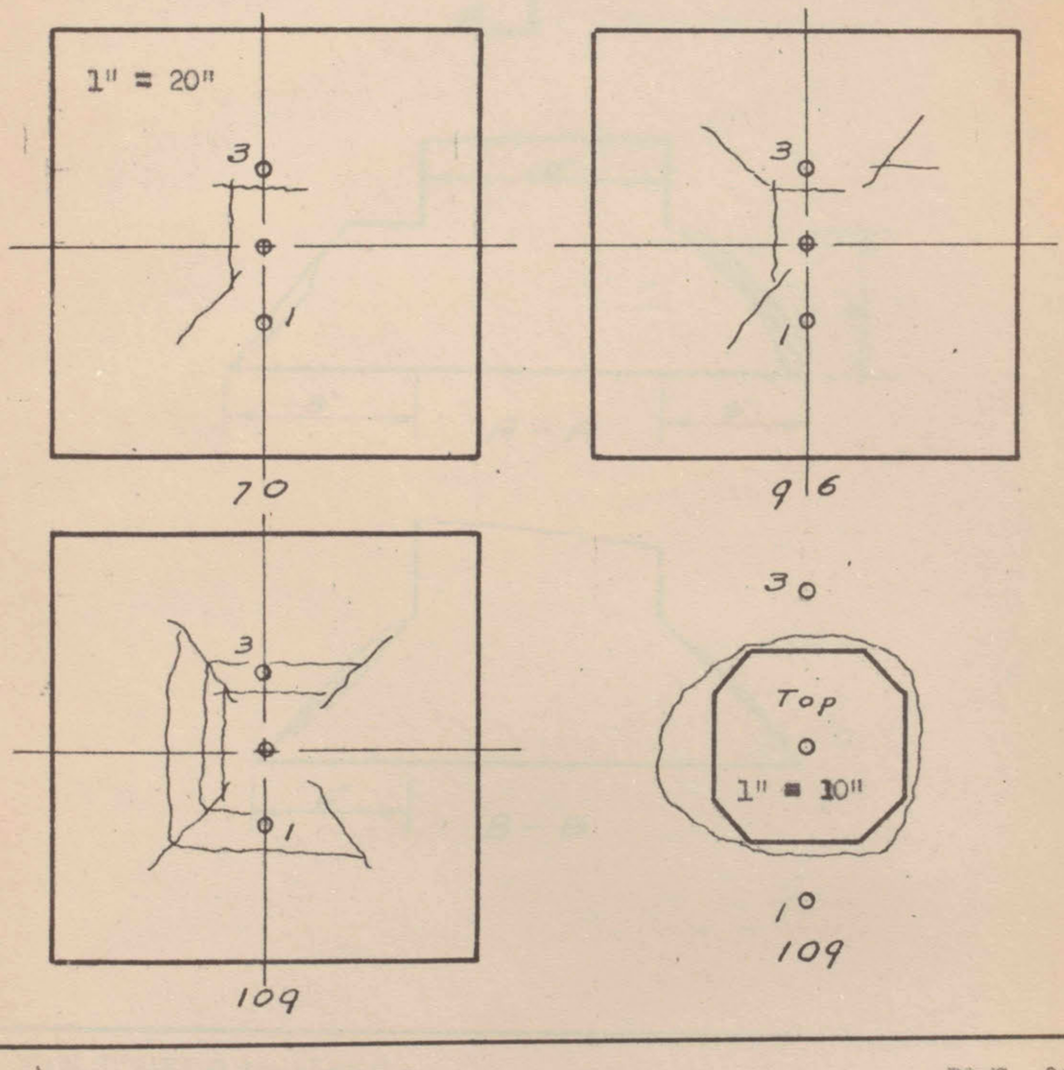


SLAB G

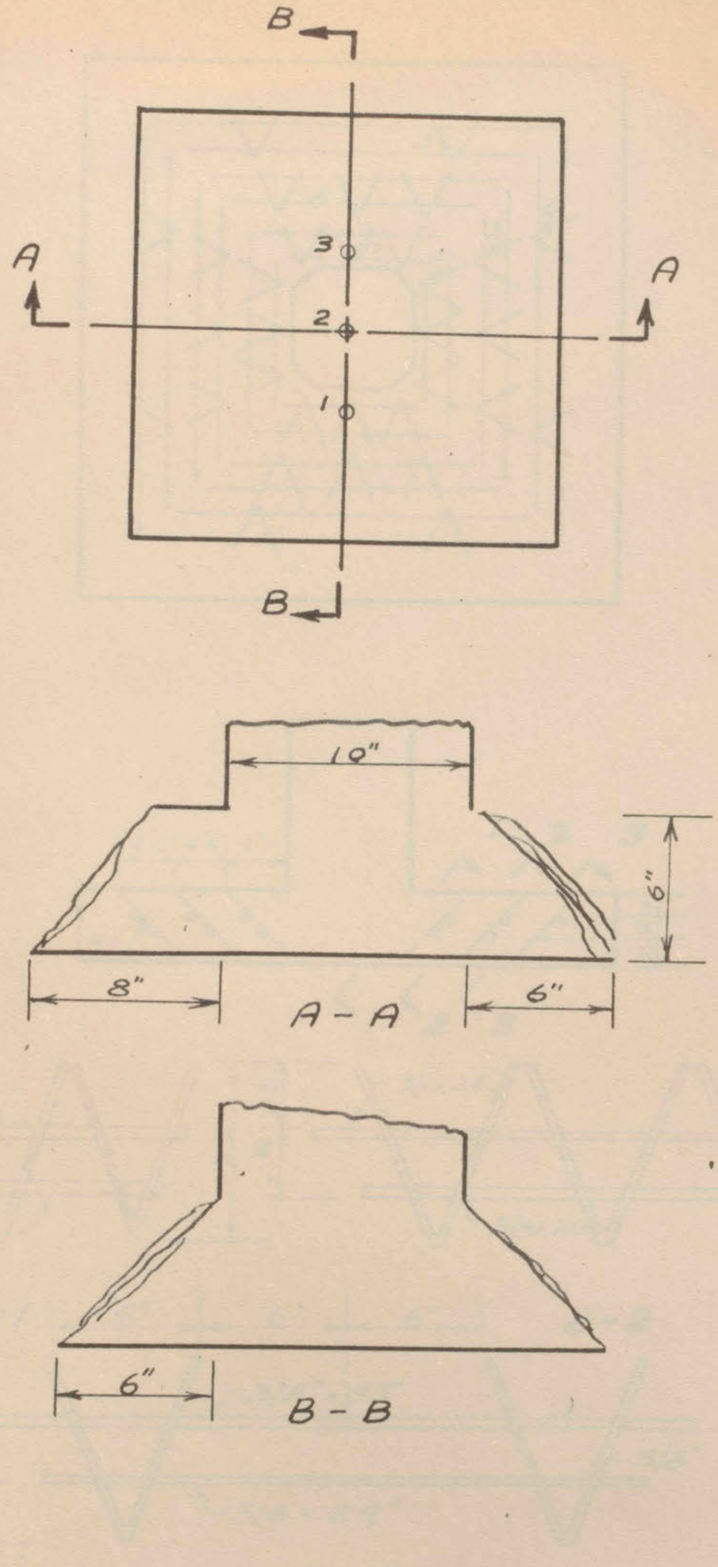
DRWG. 34



POSITION OF DEFLECTION GAGES DRWG. 35

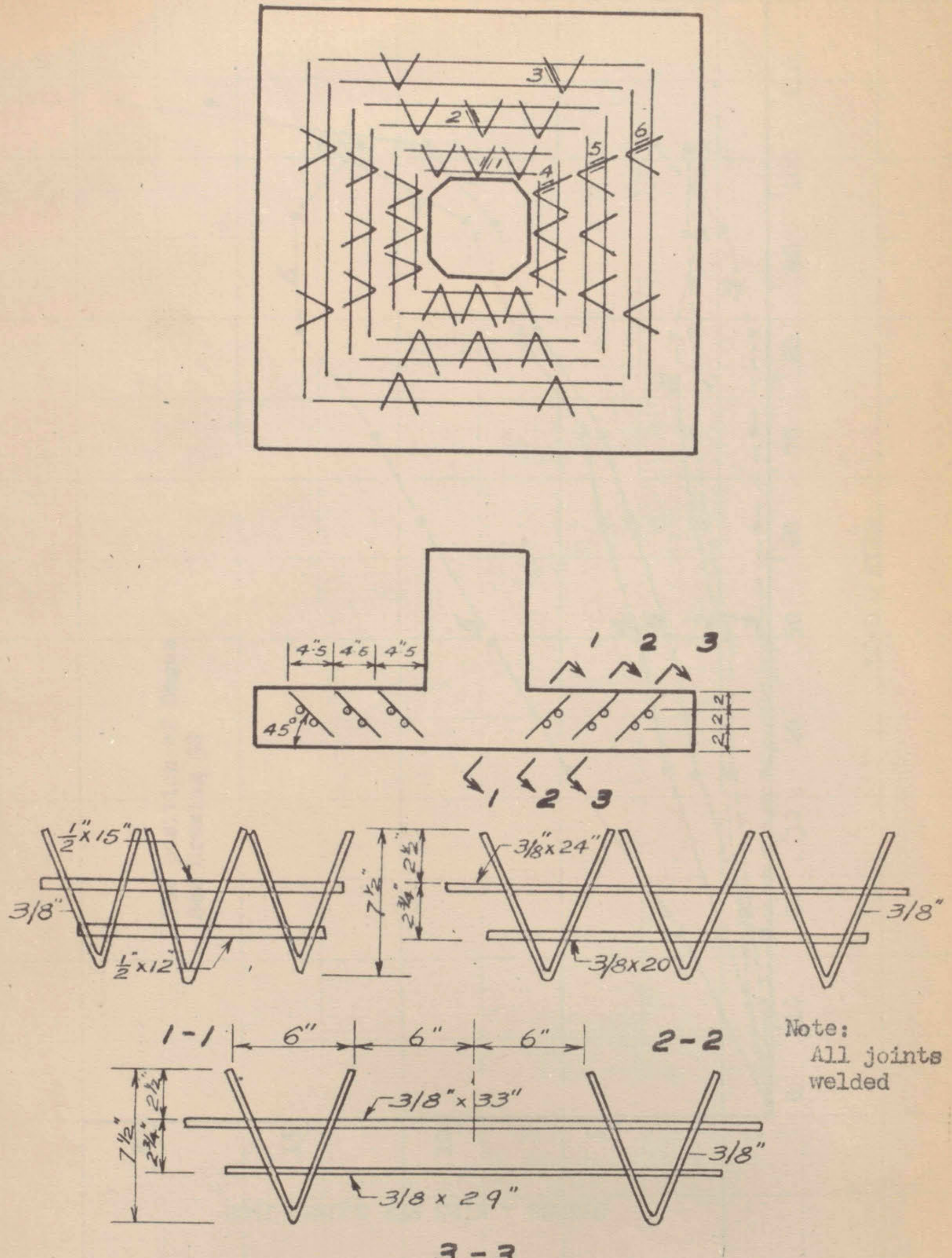


TEST PHENOMENA OF SLAB G DRWG. 36



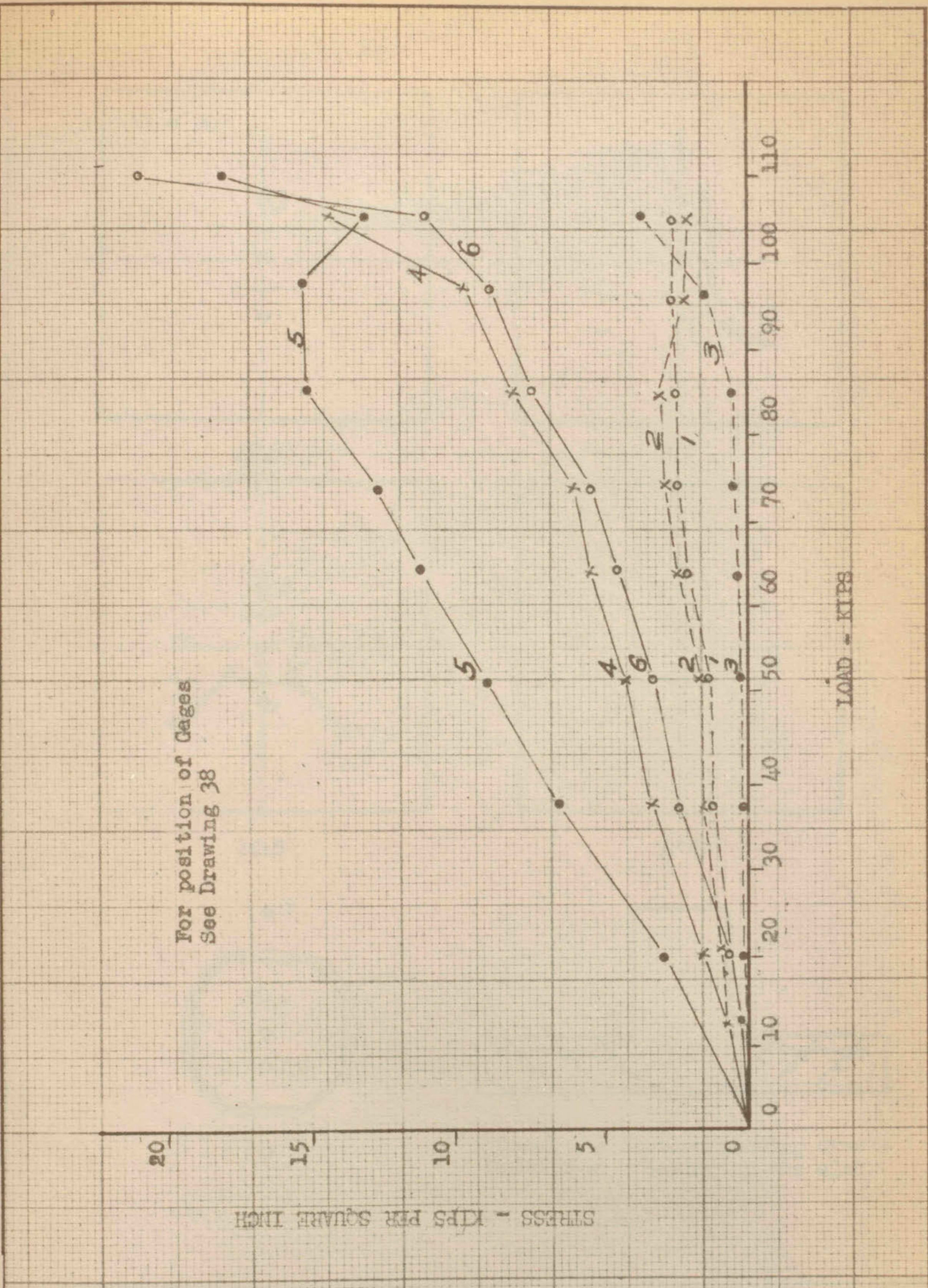
UPON BREAKING UP SLAB G

DRWG. 37



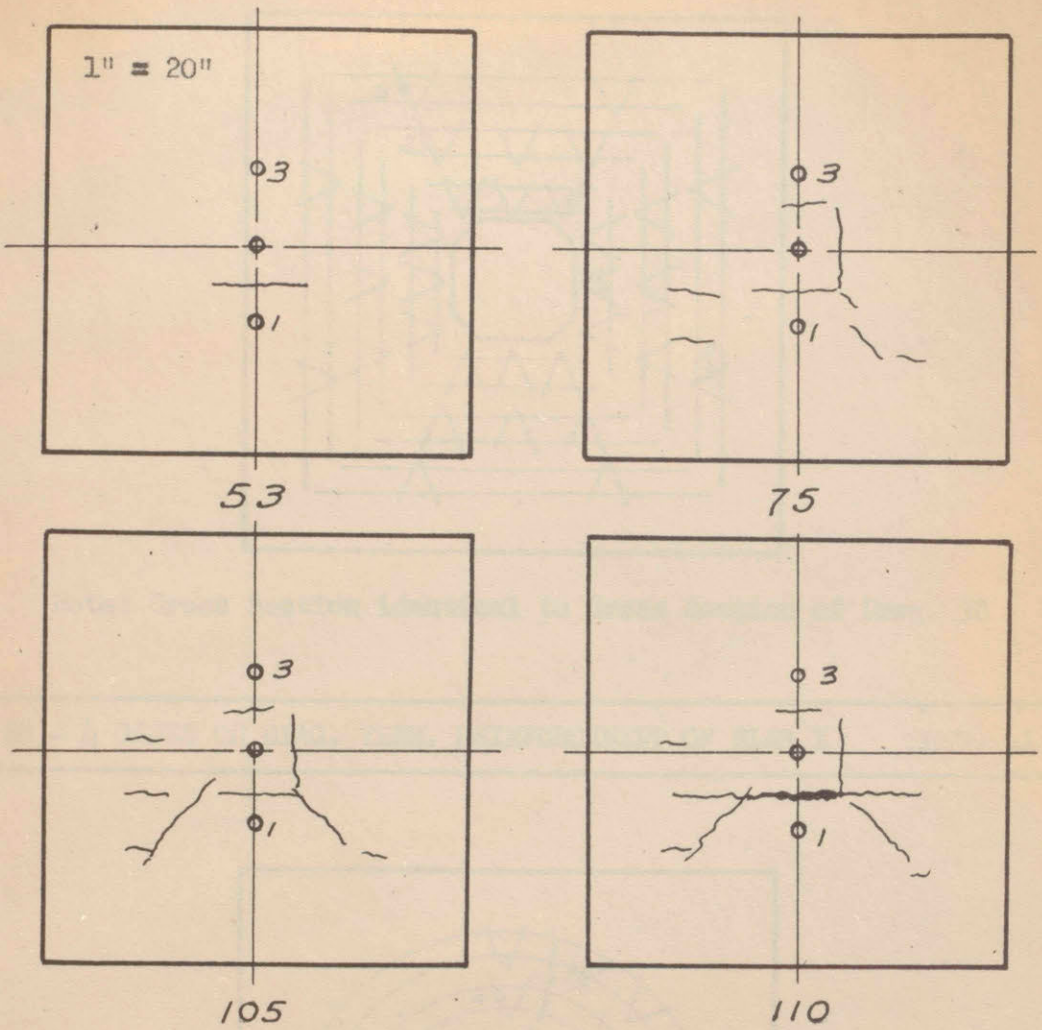
DIAGONAL TENSION REIFORCEMENT IN SLAB H

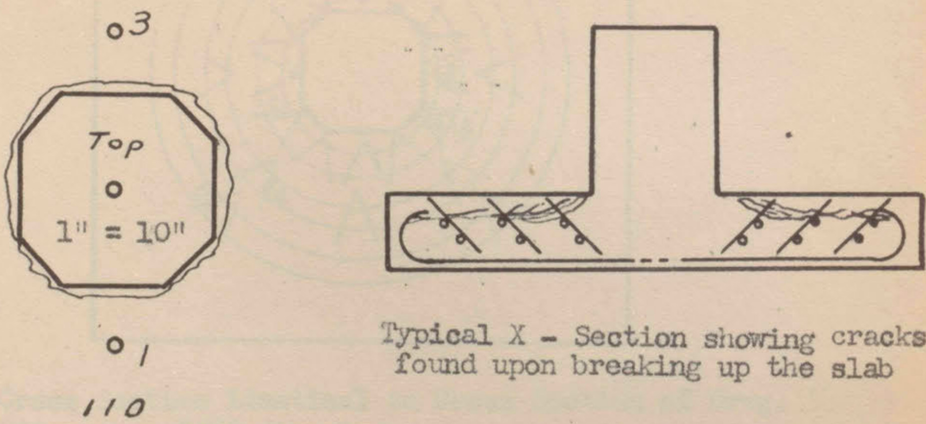
DRWG. 38



STRAIN GAGE READINGS FOR SLAB H

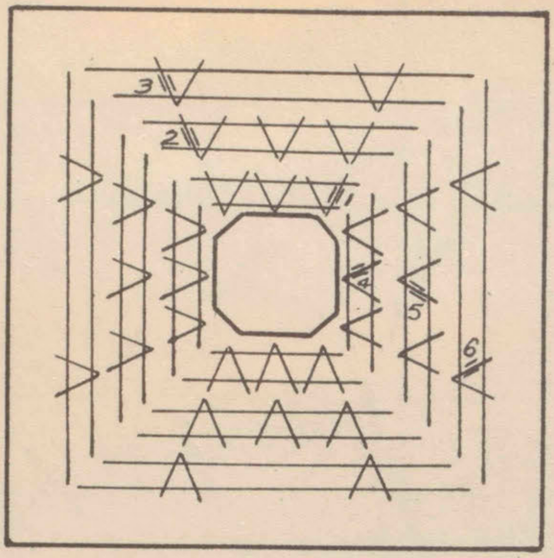
DRWG. 39





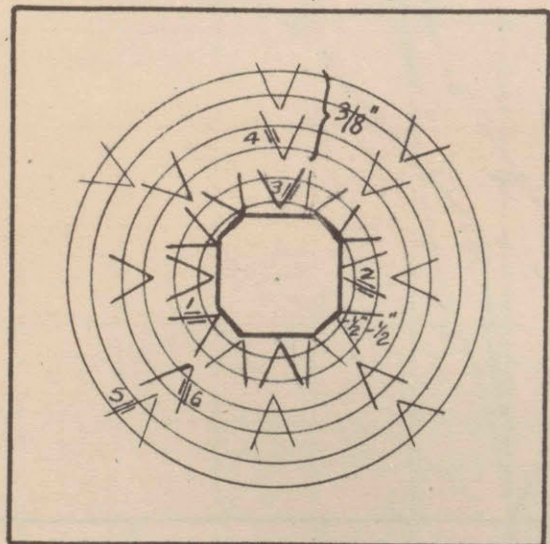
Typical X - Section showing cracks found upon breaking up the slab

TEST PHENOMENA OF SLAB H DRWG. 40



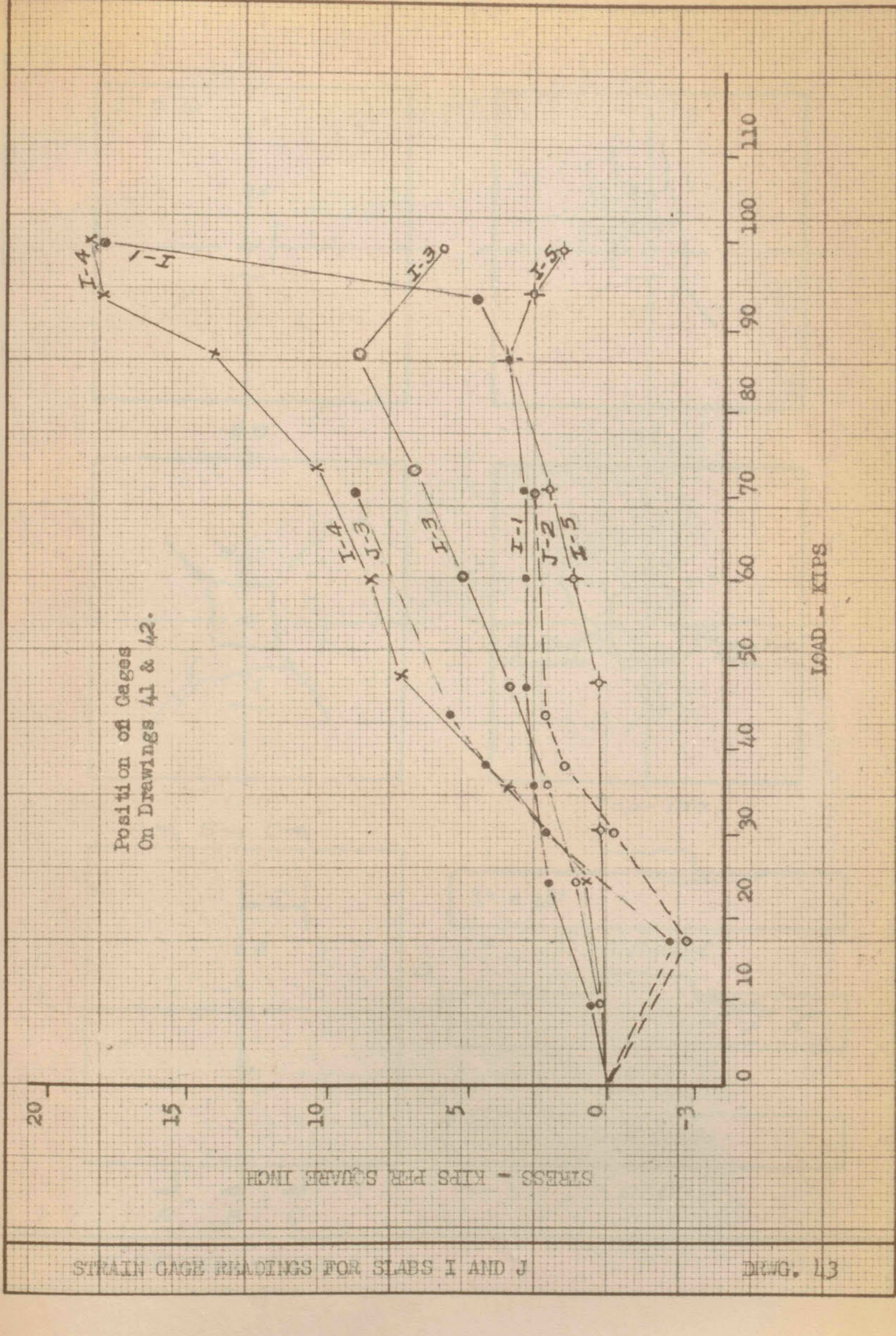
Note: Cross Section identical to Cross Section of Drwg. 38

SR - 4 GAGES ON DIAG. TENS. REINFORCEMENT OF SLAB I DRWG. 41

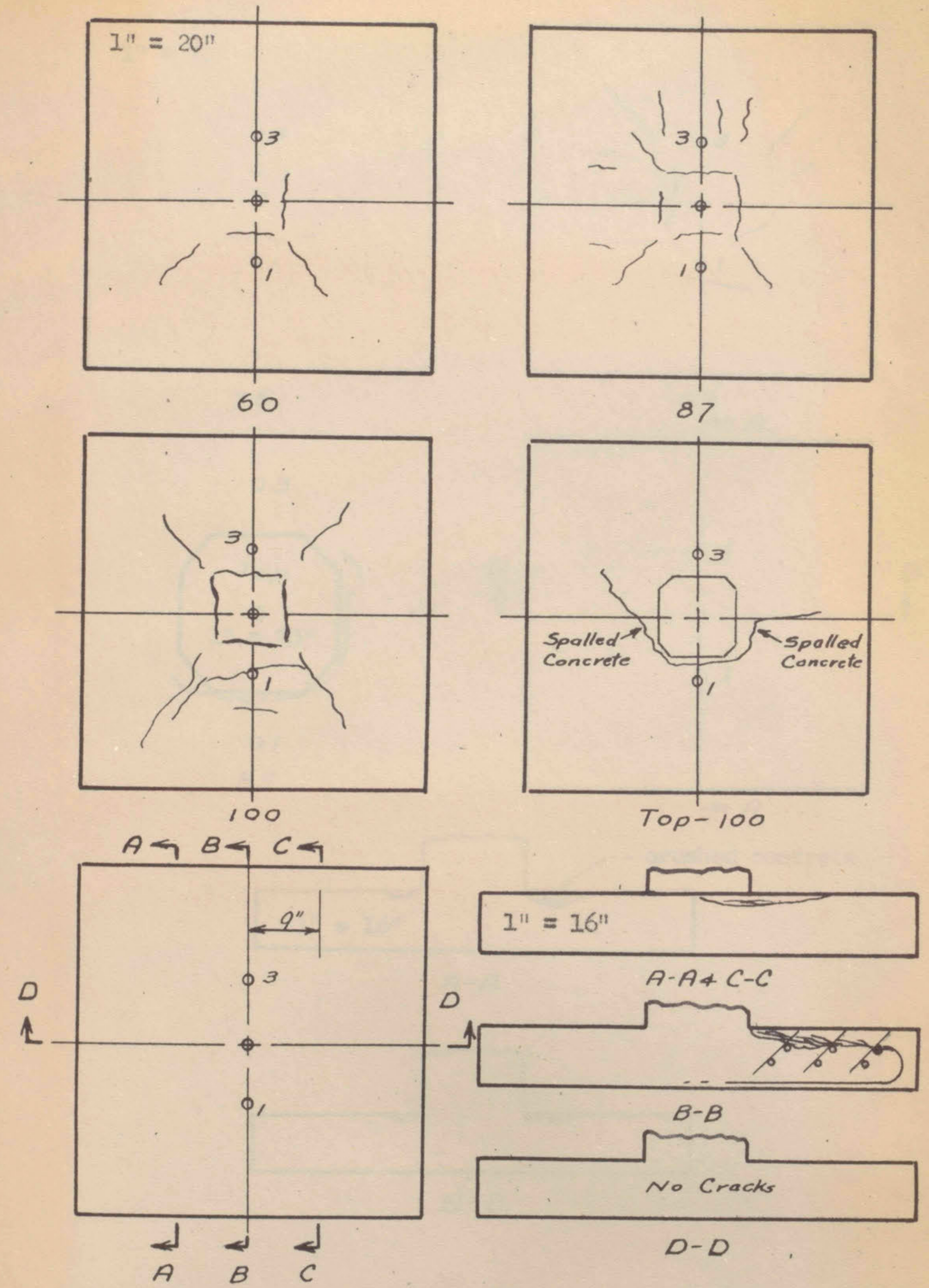


Note: Cross Section identical to Cross Section of Drwg. 38
Stirrups - 3/8" diamater

SR - 4 GAGES ON DIAG. TENS. REINFORCEMENT OF SLAB J DRWG. 42

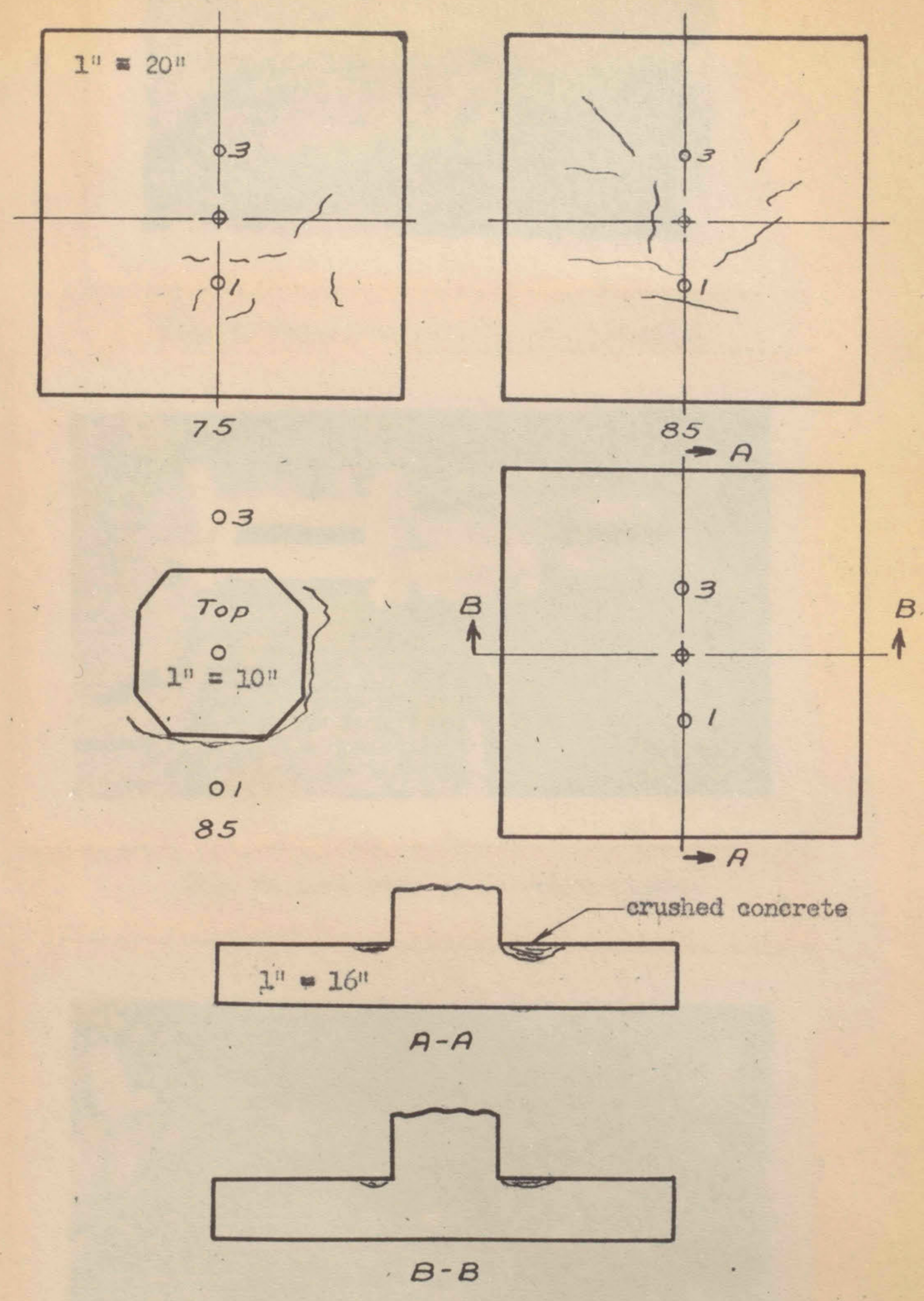


Position of Gages
On Drawings 41 & 42.
I-4
I-1
I-3
I-4
J-3
I-3
I-1
J-2
I-5
J-5
LOAD - KIPS
0
10
20
30
40
50
60
70
80
90
100
110
STRESS - KIPS PER SQUARE INCH
20
15
10
5
0
-3
STRAIN GAGE READINGS FOR SLABS I AND J
DRWG. 43



TEST PHENOMENA OF SLAB I

DRWG. 44



TEST PHENOMENA OF SLAB J

DRWG. 45

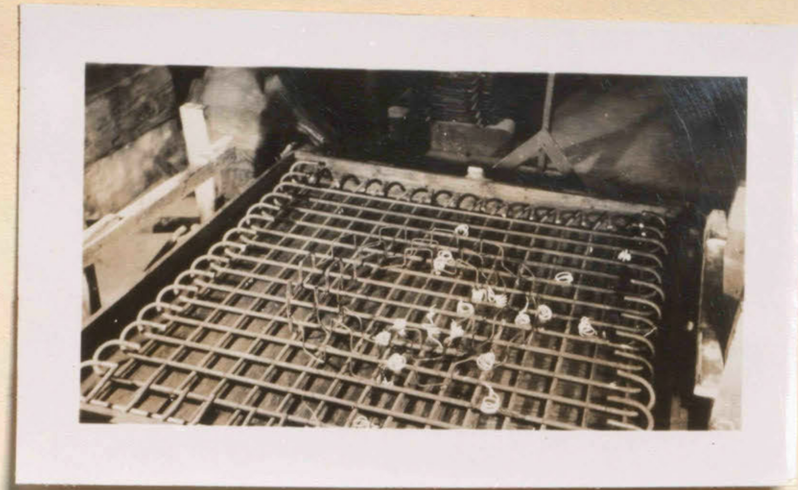


Fig. A: Gersovitz stirrup reinforcement

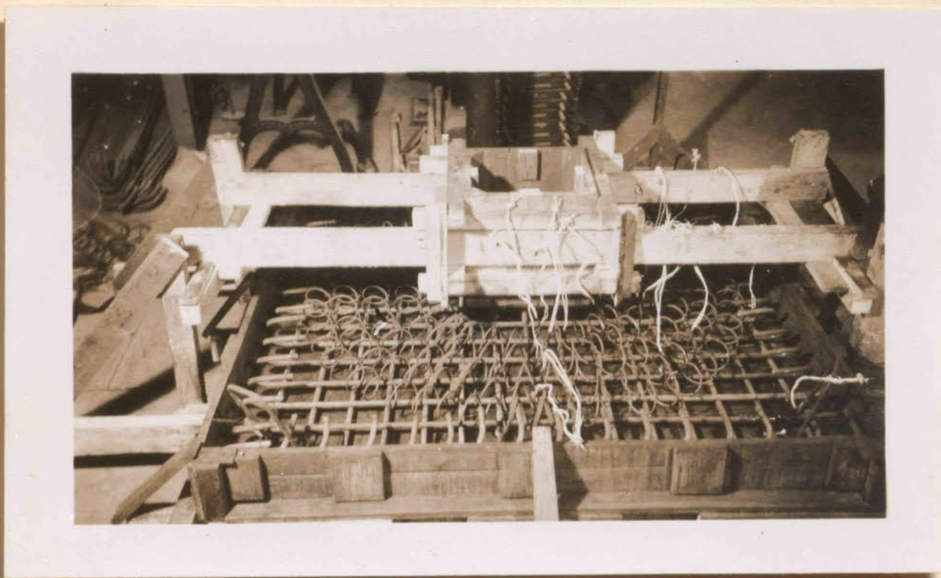


Fig. B: Gersovitz helix reinforcement

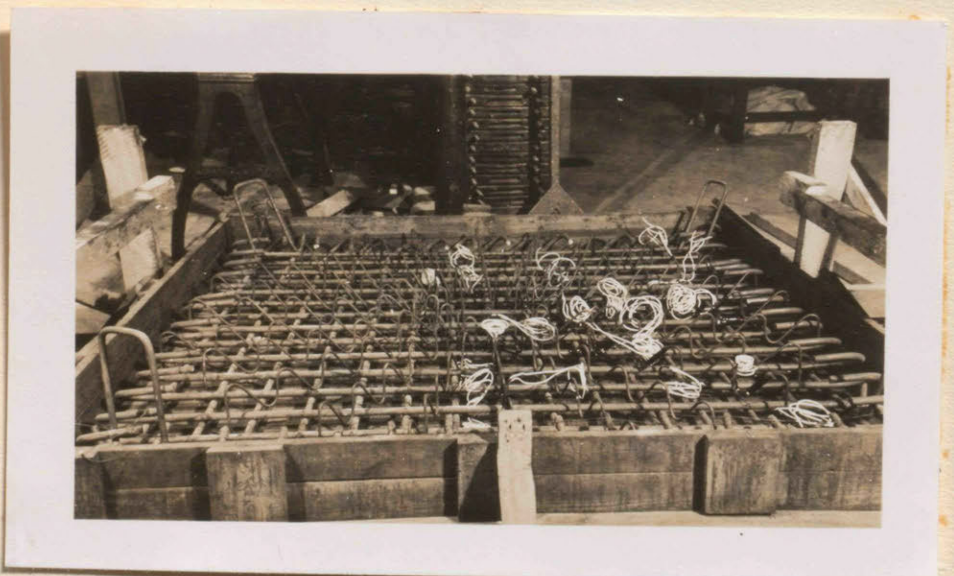


Fig. C: Gersovitz N - shaped reinforcement

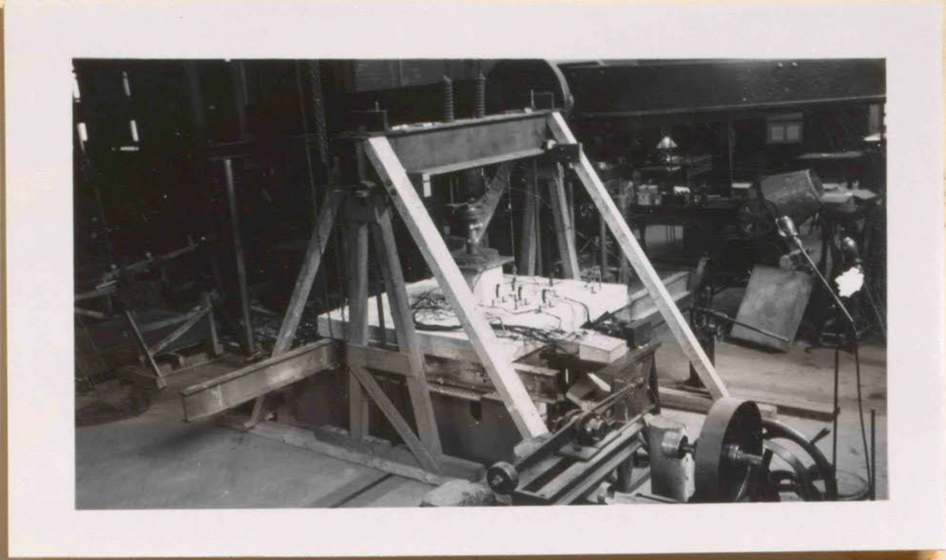


Fig. D
Gersovitz slab in testing machine

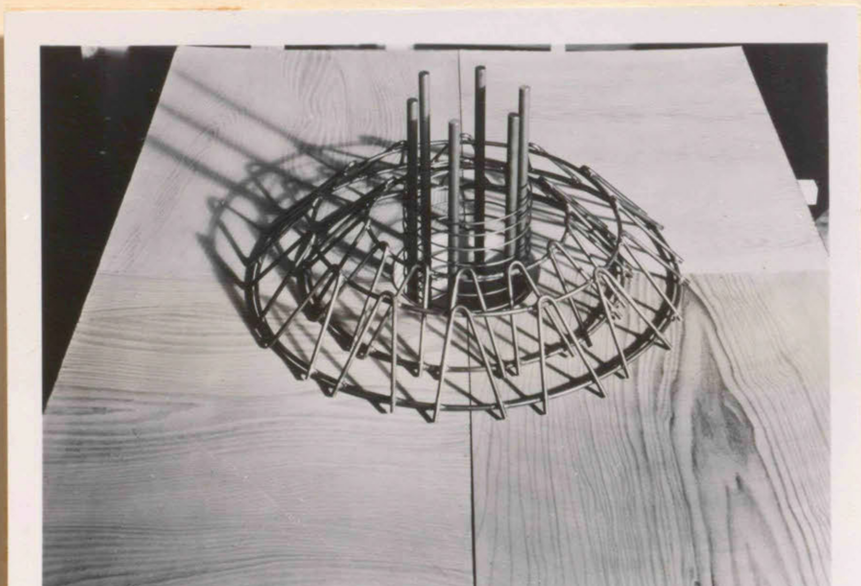


Fig. E
R/C 22 diagonal tension shearhead reinforcement

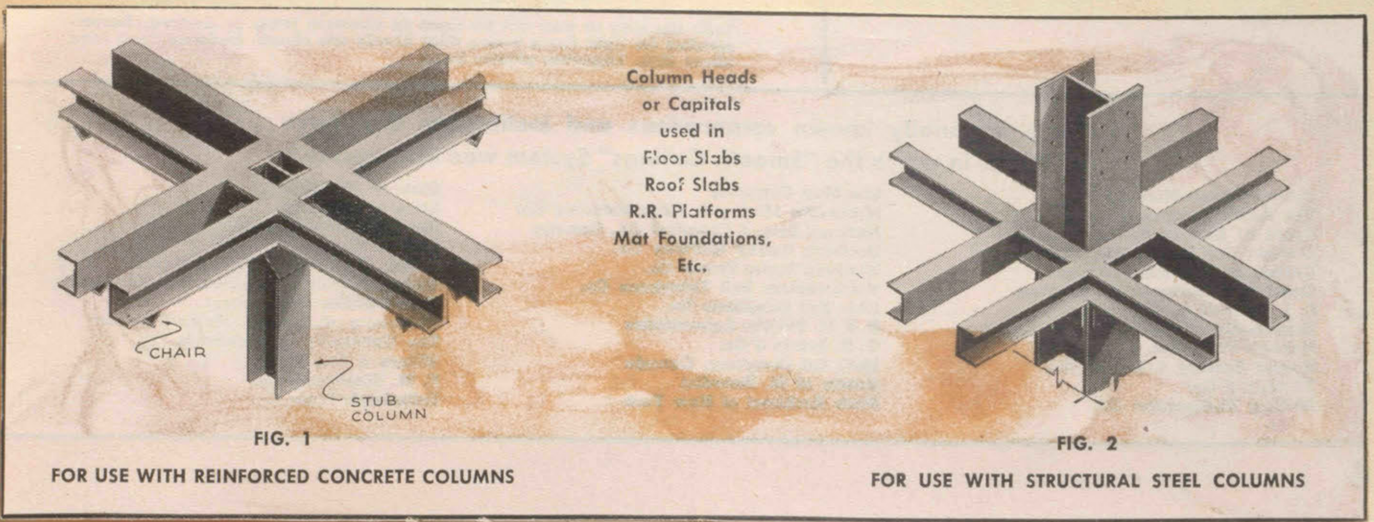


Fig. F
Wheeler grillage

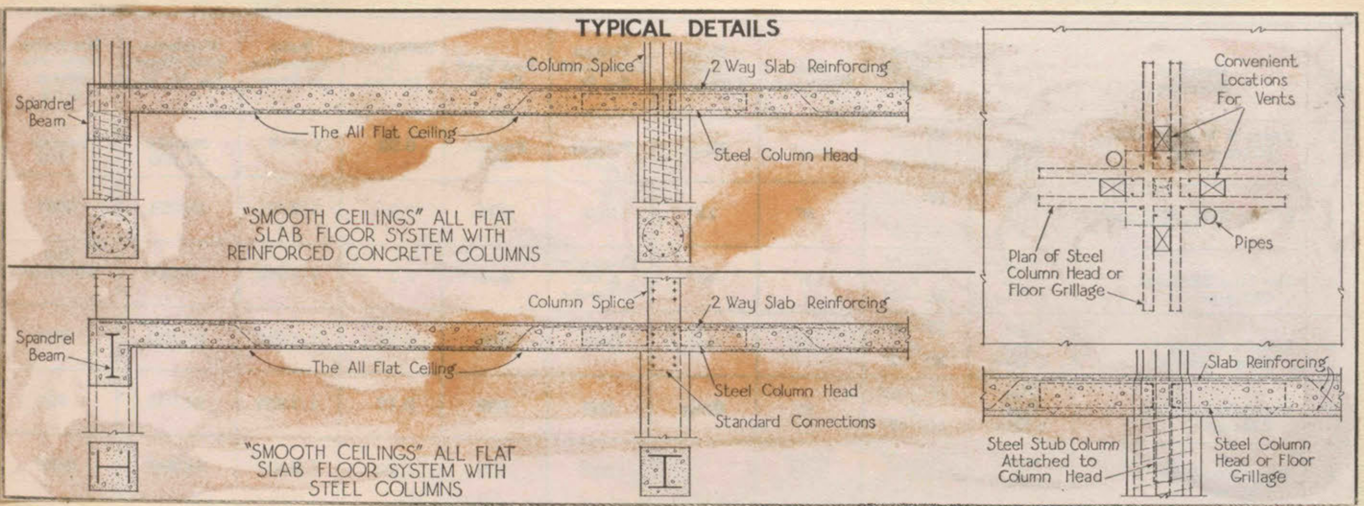


Fig. G
Wheeler grillage

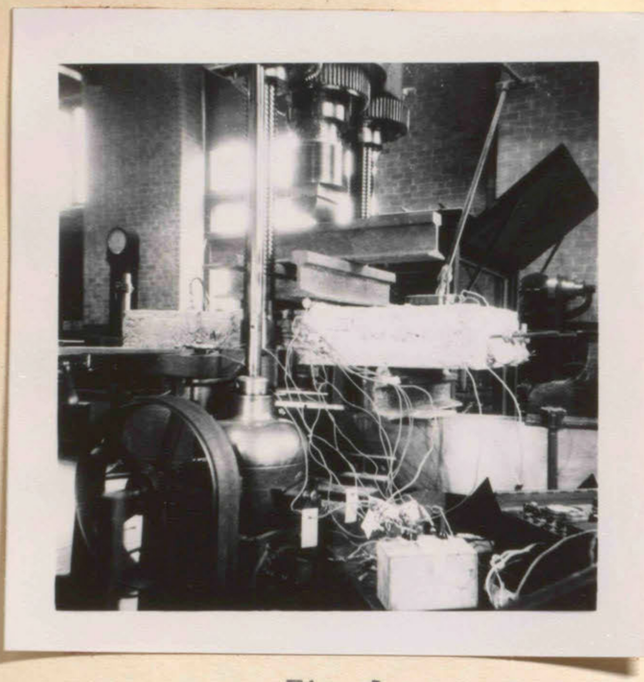


Fig. 1
Beam B - 1 in testing machine

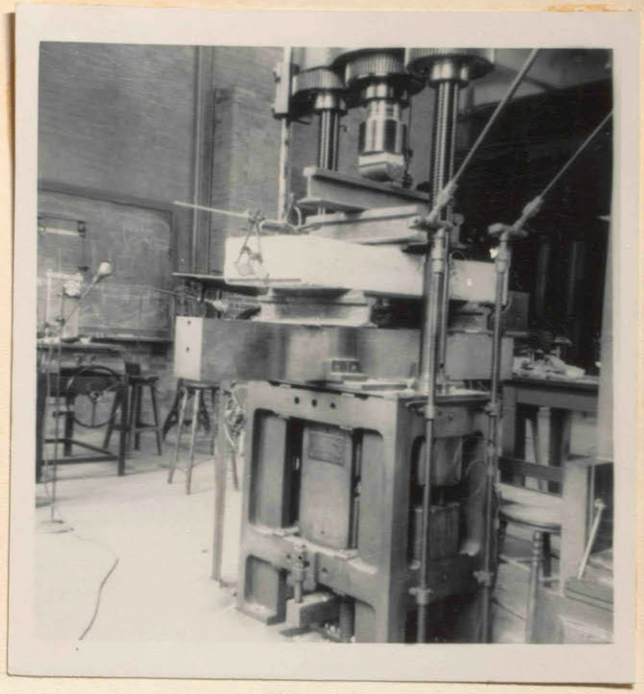


Fig. 2
Beam B - 2 in testing machine

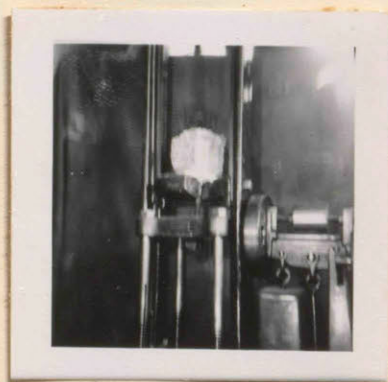


Fig. 3
Before failure

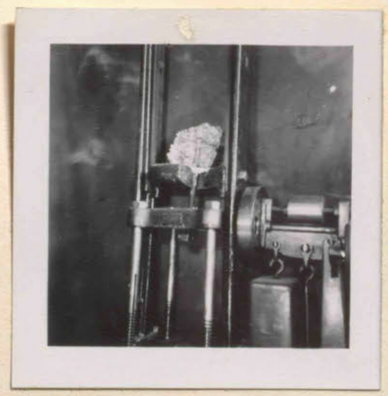


Fig. 4
After failure

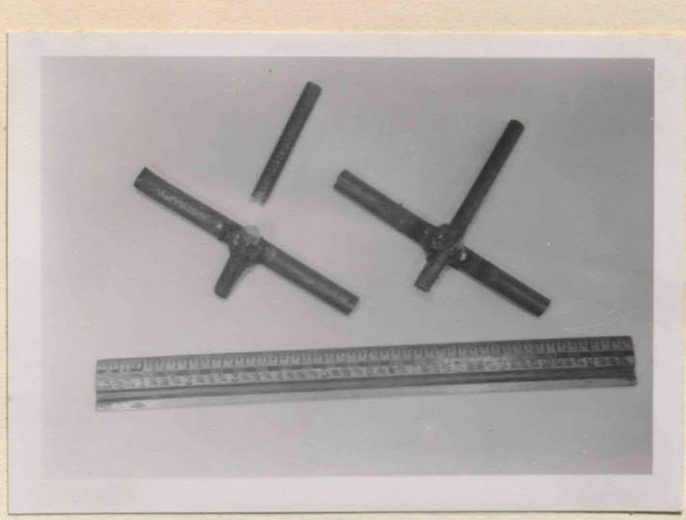


Fig. 5
Weld specimen - before and after failure

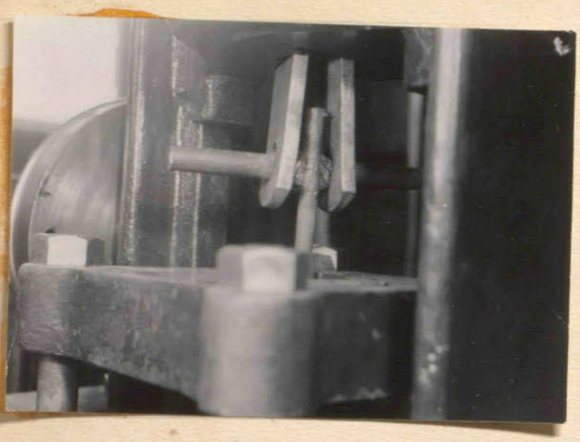
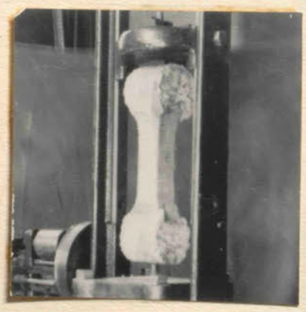


Fig. 6
Weld strength specimen

Fig. 7
Plane of failure of beam A - 1



Figs. 8
Tension test specimen

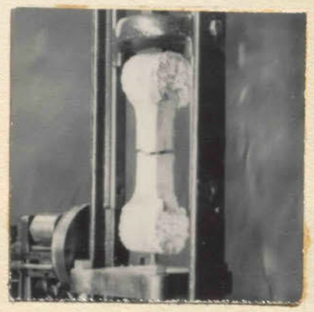
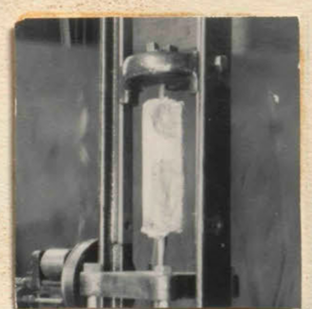
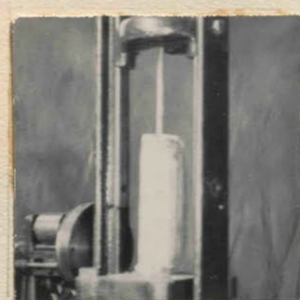


Fig. 9
After failure





Figs. 10
Before and after bond failure

Fig. 11
Bond gages for beams C, D, E, & F.

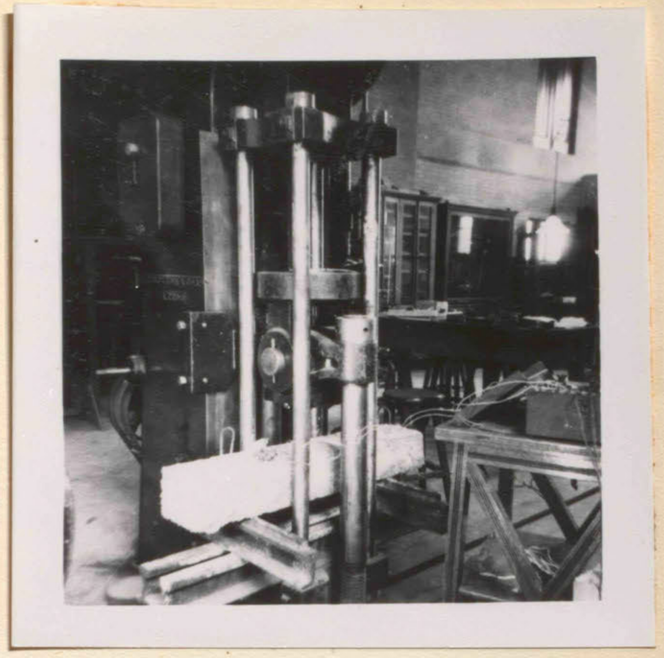


Fig. 12
Beam in testing machine - beams C, D, E & F

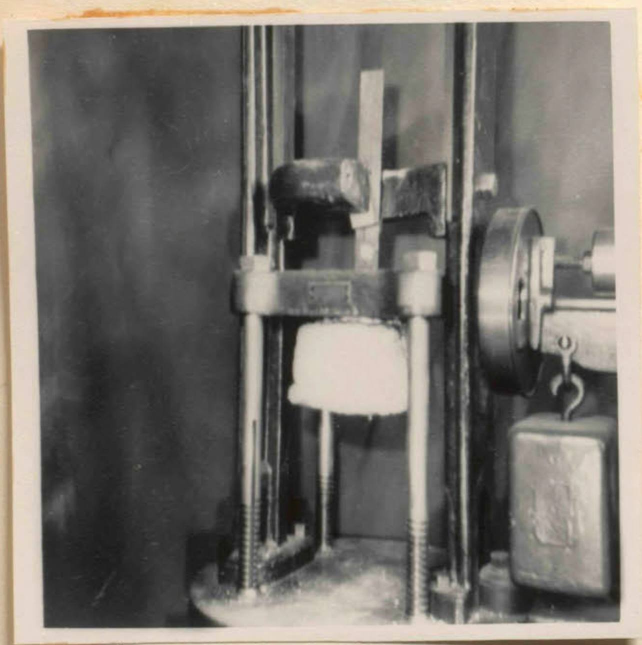


Fig. 13
Bond test specimen for beams E - 1 and E - 2

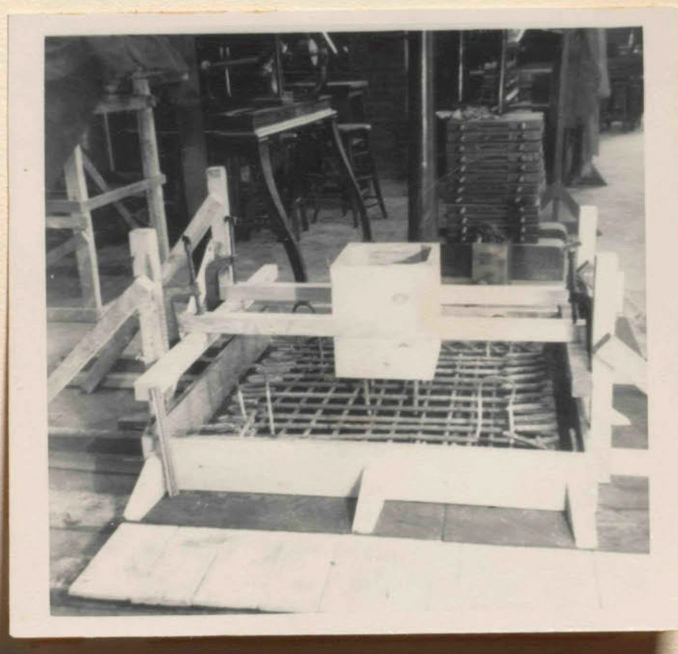


Fig. 14
Slab G

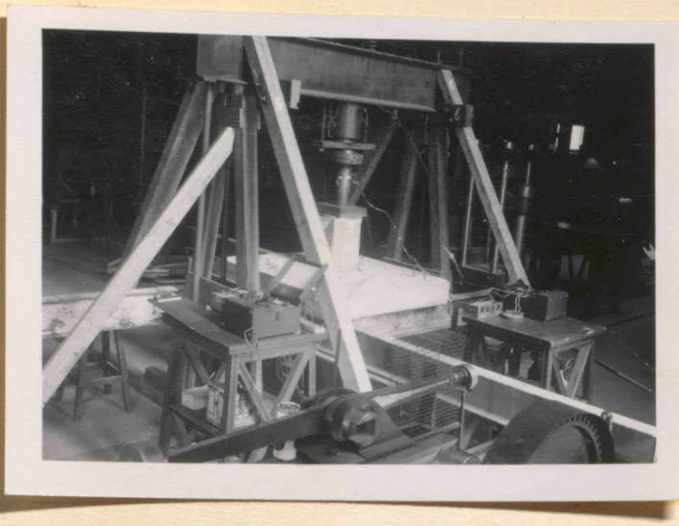


Fig. 15
Slab specimen in testing machine

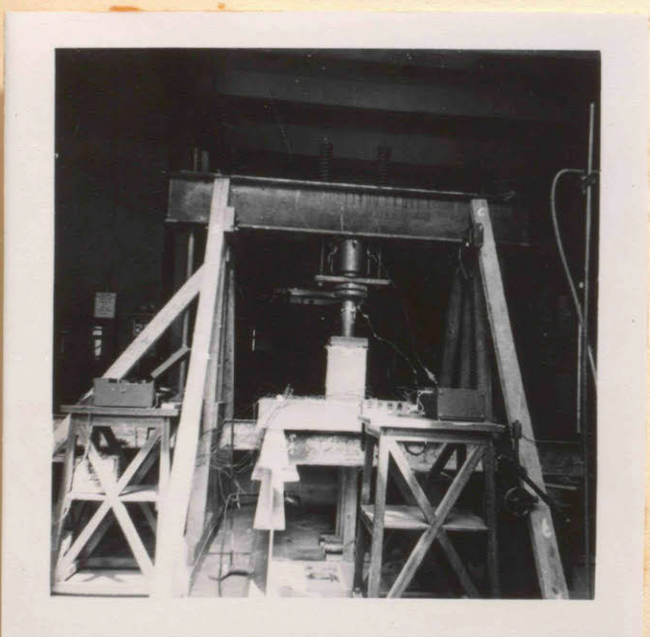


Fig. 16
Slab specimen in testing machine

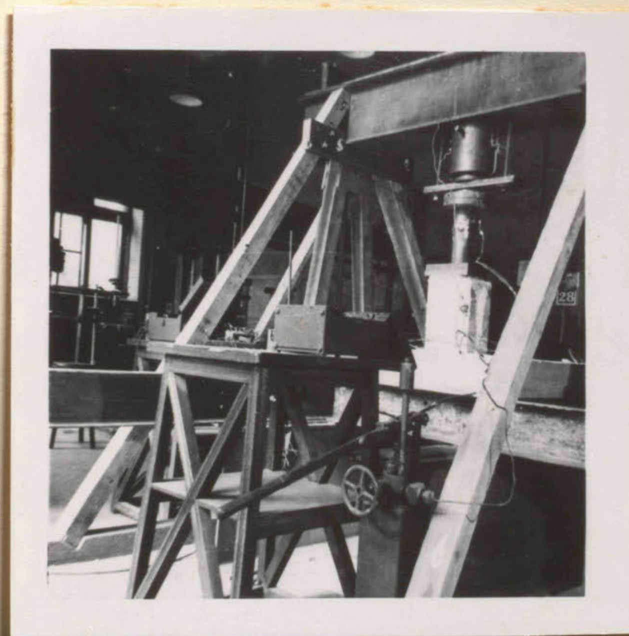


Fig. 17
Hydraulic pump

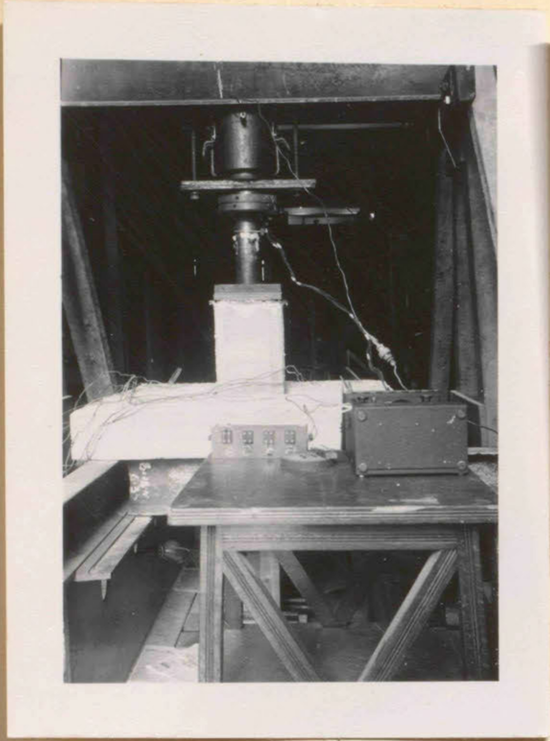


Fig. 18
Loading block

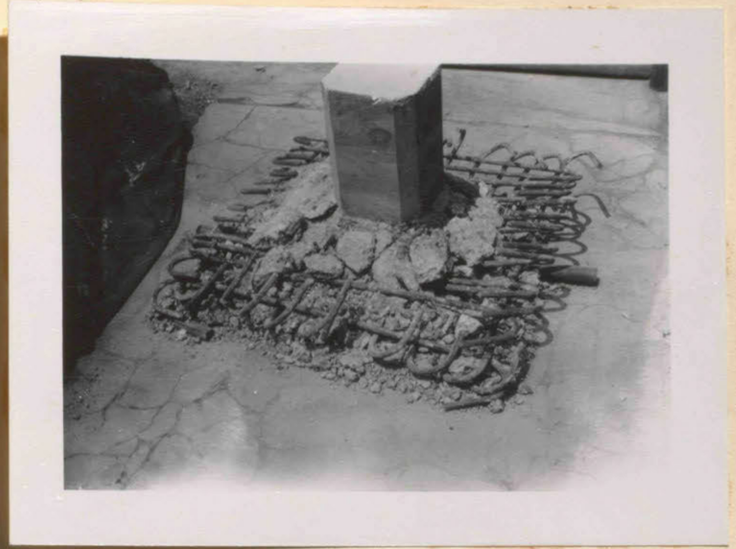


Fig. 19
Slab G - broken up

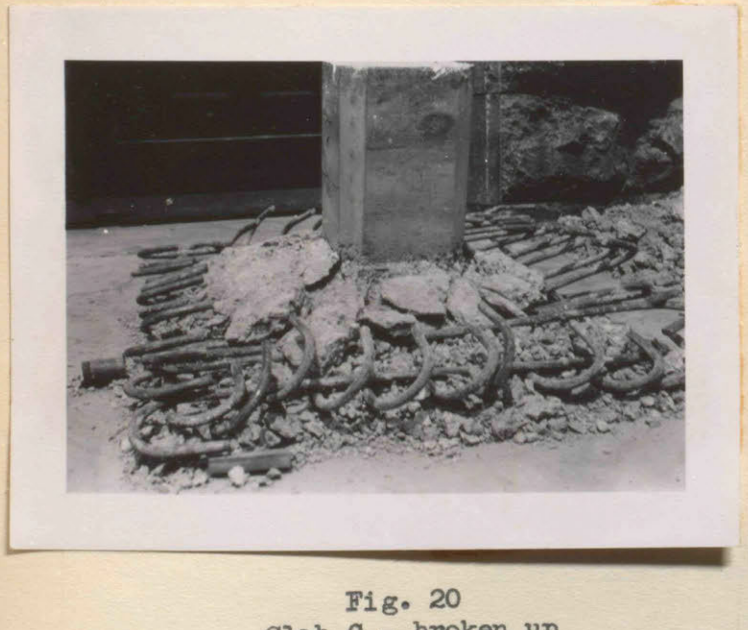


Fig. 20
Slab G - broken up

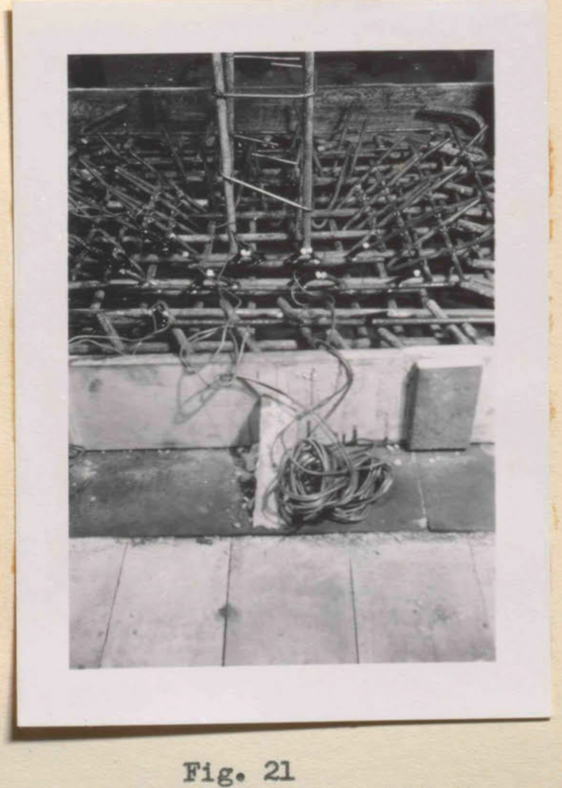


Fig. 21
Reinforcement for slabs H & I

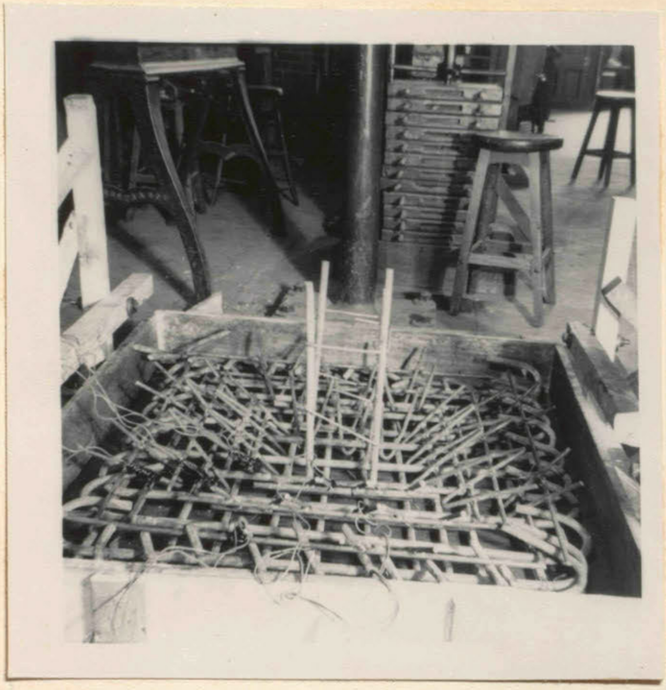


Fig. 22
Reinforcement for slabs H & I

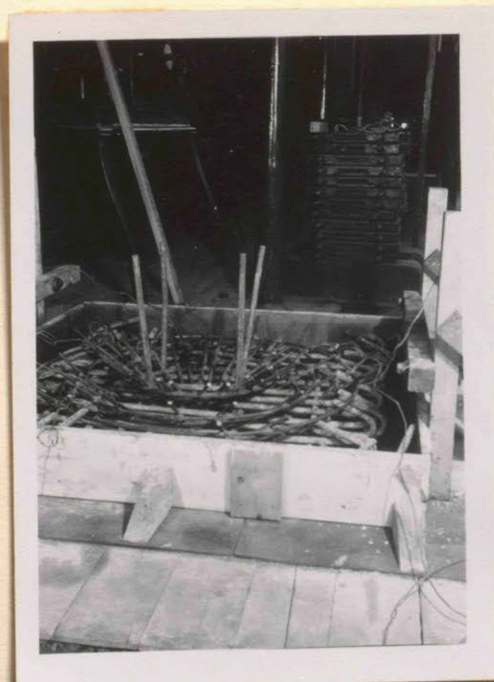


Fig. 23
Reinforcement for slab J