

CHAPTER VII CONCRETE SLABS REINFORCED WITH HIGH YIELD POINT STEEL BARS

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SYNOPSIS

Four groups of 34 in by 8 ft slabs were tested to study the effect of variation in yield point, combination of high strength concrete with steels of high and low yield point, effect of concretes of different strengths in combination with steel of constant yield point and effect of varying span length when using steel and concrete of constant strength

Concentrated loads were applied through hydraulic jacks so as to produce moments and shears simulating those caused by uniform loads

In these tests no difference in behavior between rail steel and billet steel was found. Among the significant phenomena noted were with low steel ratios the ultimate strength of the steel was developed at loads not greatly differing from relative yield points or ultimate strengths, with intermediate steel ratios secondary negative compression failure took place, high steel ratios prevented secondary compression failure. Apparently the effective steel ratio has more to do with the strength of a slab than the crushing strength of the concrete.

The test program, had for its object the investigation of the behavior of concrete slabs reinforced with steel bars of various grades

In these tests the effect upon the slab strength of each of four variables was studied in separate groups as follows:

Group 1D Effect of variation in yield point

Group 2D Combination of high strength concrete and steels of high and low yield points

Group 3D Effect of concretes of different strengths in combination with steel of constant yield point

Group 4D Effect of varying the span length, when using steel and concrete of constant strengths

Details are given in Table 1. It was also proposed to observe if the behavior of rail steel differed in any way from that of billet steel after it had once been embedded in the concrete.

THE SPECIMENS

The span of all the continuous slabs was made 8 ft between supports, and a width of 34 in for all slabs and a depth of 4 in for the majority was adopted. The effect of continuity was obtained in each

continuous slab by means of an overhang at each end, the length of which, 0.408L, was such that the negative moment produced by the weight of the slab was twice as great as the positive moment. The small percentage of steel in the slabs of Group 1D not only made it possible to study the influence of yield point upon slab strength, but also made it possible to compare the behavior of lightly reinforced slabs with that of slabs having more usual steel ratios. In Group 3D, the high percentage of reinforcement, in combination with concretes of various strengths, made possible a preliminary study of what constitutes a satisfactory commercial slab. In the slabs of Groups 2D and 4D the percentage of reinforcement was closely that in common practice. In order approximately to maintain this percentage, the thickness of slab 4D3 was made 5 in and that of slab 4D4 was made 6 in.

The slabs of Groups 1D, 2D and 4D were cast and tested indoors. Those of Group 3D were cast outdoors in May and tested outdoors in June, 1936. The dates of casting were so chosen that each slab should be 28 days old when tested. - Actually one or two slabs were as old as

33 days, but this did not materially affect results. All slabs were left in the forms and covered with wet burlap for one week, after which they were allowed to stand in the open air. While test cylinders, stored under the same conditions showed some variation from the design concrete strength, the variation was not great and was always on the side of greater strength than anticipated.

the accuracy of regulation and ease of control, a Lancaster mixer was used.

In the simple spans all bars were carried full length 1 in from the lower surface. In the continuous spans all bars were bent up at the points of contraflexure and carried to the extreme ends of the slabs 3 in from the lower surface. Slight variation in the placement of steel and depth of slab have been allowed for

TABLE 1

Group	Slab No	To Investigate	Type	Span L	Nominal Slab Thick In	Nominal Conc Str p s i	Kind of Steel	Nominal Y P Steel kip/sq in	No of Bars	Size of Bars	A _s	%	Equivalent* %	Nominal n	
1D	1D1	Effect of Y P	Continuous	8'-0"	4	3000	Billet	40	7	1/2φ	34	34	34	10	92
	1D2			"	4	3000	Billet	50	7	"	34	34	43	10	92
	1D3			"	4	3000	Rail	65	7	"	34	34	55	10	92
	1D4			"	4	3000	Billet	80	7	"	34	34	68	10	92
	1D5			"	4	3000	Billet	100	7	"	34	34	85	10	92
2D	2D1	High f' _c and different Y P's	Continuous	8'-0"	4	6000	Billet	40	7	3/8φ	77	76	76	5	92
	2D1X			"	4	3000	Billet	40	7	"	77	76	76	10	89
	2D2			"	4	6000	Billet	100	7	"	77	76	1	93	5
3D	3D1	Effect of Conc Strength	Continuous	8'-0"	4	2000	Rail	65	7	3/8φ	2 15	2 11	3 43	15	82
	3D2			"	4	3000	Rail	65	7	"	2 15	2 11	3 43	10	84
	3D3			"	4	4500	Rail	65	7	"	2 15	2 11	3 43	7 1/2	86
	3D4			"	4	6000	Rail	65	7	"	2 15	2 11	3 43	5	88
4D	4D1	Effect of Span Length	Simple	5'-0"	4	3000	Rail	65	7	3/8φ	77	76	1 24	10	89
	4D2			8'-0"	4	3000	Rail	65	7	3/8φ	77	76	1 24	10	89
	4D3			12'-0"	5	3000	Rail	65	5	1/2φ	98	72	1 17	10	90
	4D4			16'-0"	6	3000	Rail	65	6	1/2φ	1 18	69	1 12	10	90

* Equivalent in total tension, at yield point, to this % of steel having yield point of 40,000 p s i

The coarse aggregate was Delaware river gravel of 1/4 in nominal size, and had a fineness modulus of 5.40. The sand came from the same source, its fineness modulus being 2.93. Both sand and gravel were donated by The Warner Bros Company of Wilmington, Delaware. Lehigh portland cement was used. The billet steel was furnished by the Bethlehem Steel Company and the rail steel by the Rail Steel Bar Association. To all these organizations acknowledgment for their cooperation is due. Because of

in the computations. In slab 4D1, where measurements for slip were made, one end of one bar was left without a hook, elsewhere all bars were provided with Considéré hooks of as large radius as was possible in such thin members. Complete details of all slabs are given on Figure 6.

TYPES OF LOADING

Slabs, in general, are designed to carry uniformly distributed loads, but in order to preclude the unknown arching effect

of an applied uniform load, the test slabs were subjected to a series of concentrated loads producing moments and shears simulating those caused by a uniform load. On the simply supported slabs, therefore, two concentrated loads were applied at the outer quarter points, in which position the end shears are the same as those resulting from uniform loads producing the same bending moments at midspan.

On each cantilevered slab four equal loads were employed, so spaced as to

LOADING APPARATUS

Loads were applied to the slab by means of the hydraulic apparatus shown in Figures 1 and 7. Two timber cribs supported two I beams; and on the top of each I beam was spot welded a $\frac{1}{4}$ -in. square bar, with the upper corners rounded off. A $\frac{1}{2} \times 1\frac{1}{2}$ in. steel flat prevented the $\frac{1}{4}$ -in. square from cutting into the concrete. This device, which definitely fixed the position of the reactions, carried the slab. Steel flats

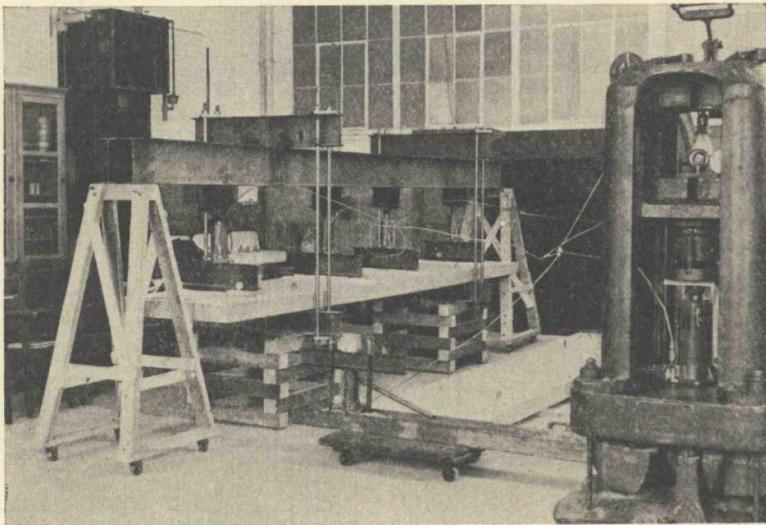


Figure 1

produce negative moments twice as great as the positive moment, and so placed as to cause the points of contraflexure to coincide with those which would have been produced by a uniform load. Here again the shears at the supports are practically the same as those accompanying uniform load. This condition is the same as that of complete fixation, as in an intermediate span of a beam of uniform cross section, continuous over many equally spaced supports, and loaded uniformly in all panels. The positions of all loads are shown on Figure 6, and key moment diagrams are given on Figure 8.

were bedded on the slabs at the load points. The flat on each cantilever was grooved; and a knife edge made up of an angle and a round spot welded together, and welded to the lower flange of a loading I beam, was fitted into each groove. (See Fig. 2 and Fig. 7). The midspan loading beams rested on 1 in. rounds placed on the steel flats.

Hydraulic jacks were placed on top of the I sections, and the rams of the jacks bore upward against short beams bolted in place on the under side of two large reaction beams. Two cross beams were bolted to the top of the reaction beams,

directly over the beams supporting the slab, and were connected to these supporting beams by means of rods passing

possible to roll the whole upper works to one side when a slab was to be placed in or removed from the machine. It was

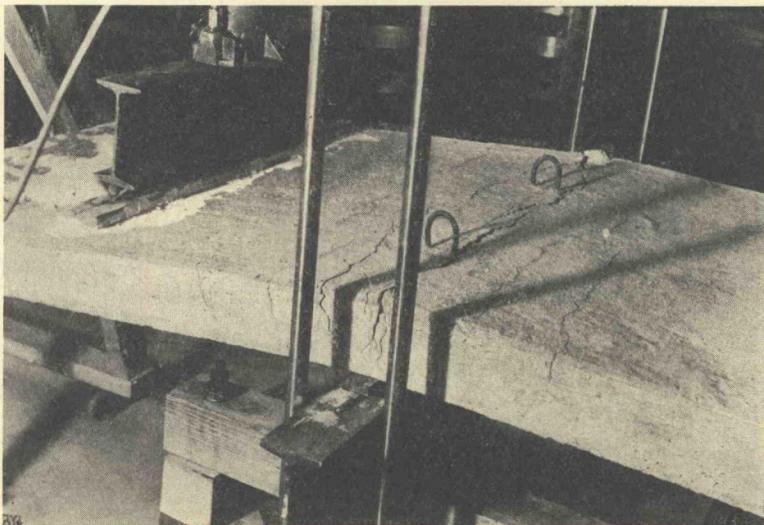


Figure 2

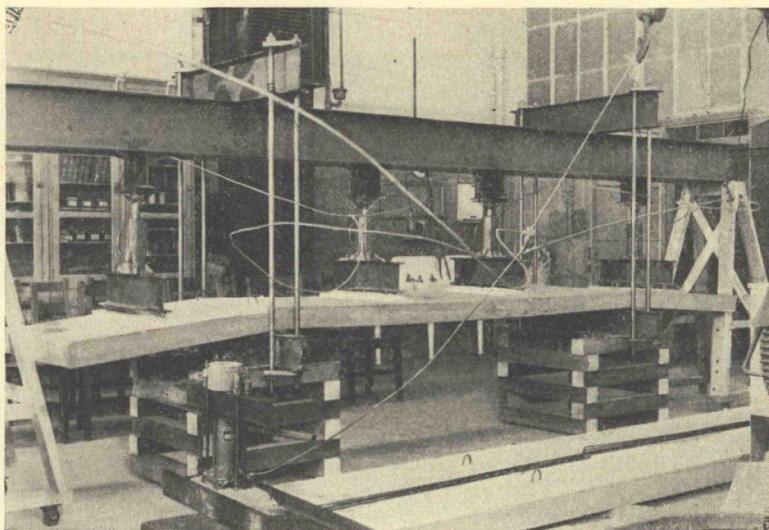


Figure 3

through cross plates. Two wooden horses, mounted on casters, supported the weight of everything above the jacks when there was no load on the slab, thus making it

possible quickly and easily to accommodate the apparatus to any slab tested.

The jacks used were commercial hydraulic jacks of 12 ton capacity. A fifth

jack, with its ram strapped down, and equipped with an auxiliary reservoir, served as a pump. A sixth jack was placed in one of the laboratory testing

slab by each loading jack could therefore be read directly on the scale beam. A heavy spring, not shown in the photographs, placed above the testing machine

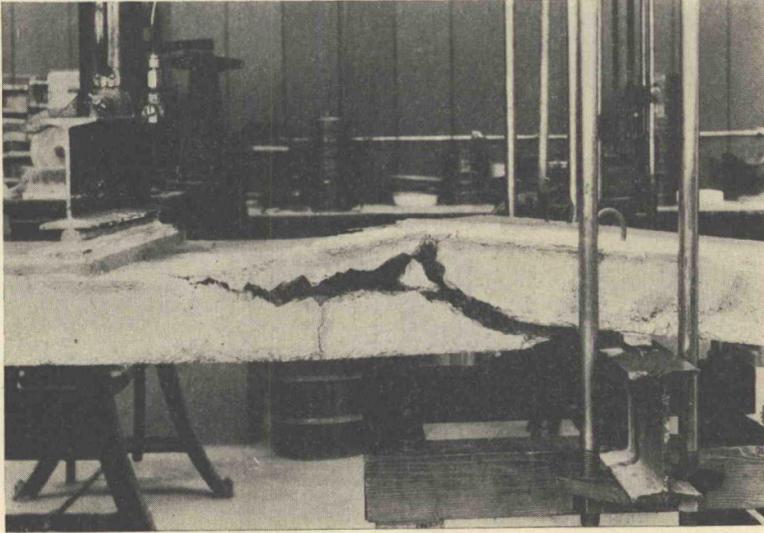


Figure 4

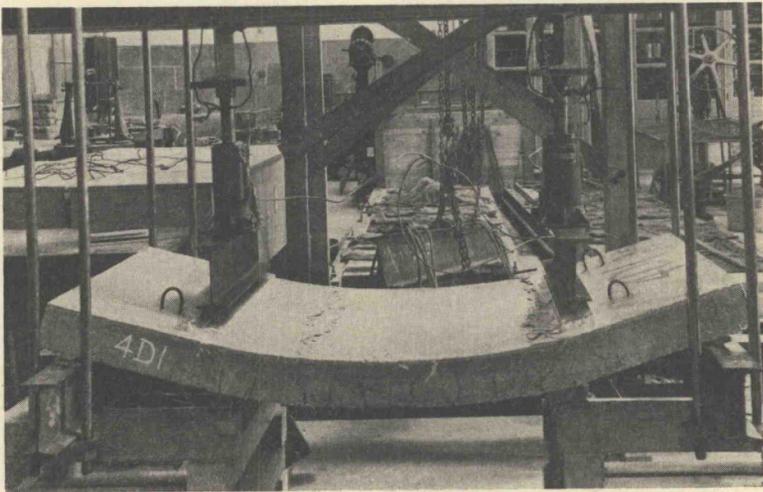


Figure 5

machines, and pressure applied at the pump was transmitted to each of the loading jacks and to the jack in the testing machine. The load applied to the

jack, made it much easier to maintain loads. Obviously it was necessary to remove the line check valve from the testing machine jack in order to allow

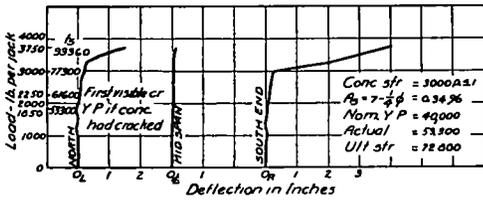


Figure 9 a
Slab 1 D 1

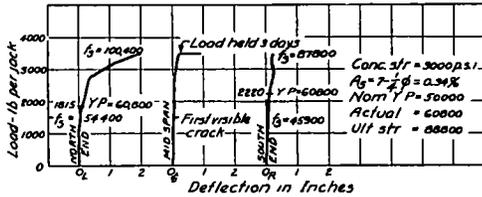


Figure 9 b
Slab 1 D 2

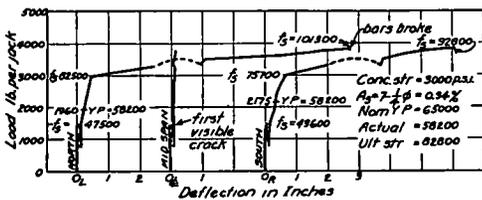


Figure 9 c
Slab 1 D 3

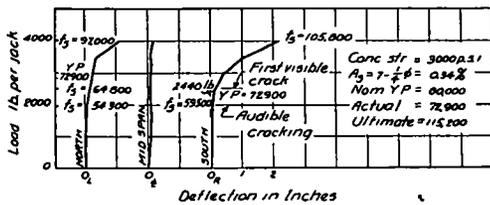


Figure 10 a
Slab 1 D 4

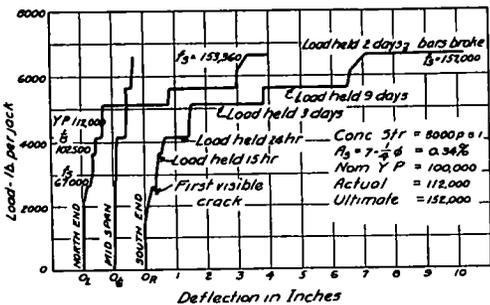


Figure 10 b
Slab 1 D 5

applied to the slab in increments commensurate with the slab capacity

No extensometer readings were taken, but, after the application of each incre-

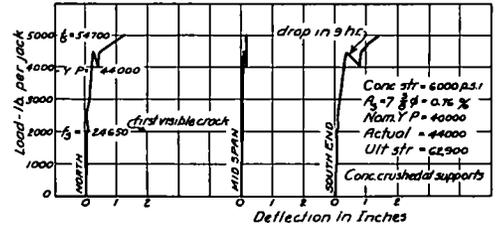


Figure 11 a
Slab 2 D 1

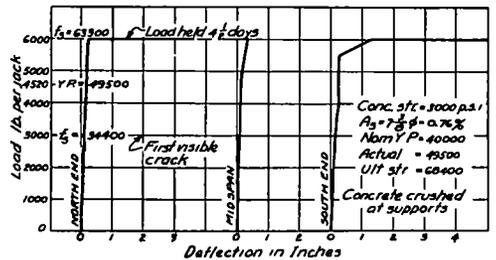


Figure 11 b
Slab 2 D 1 X

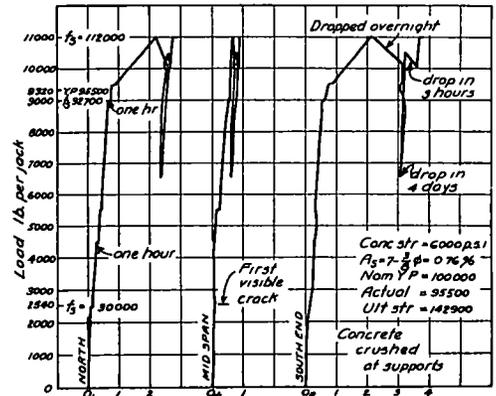


Figure 12
Slab 2 D 2

ment of live load, a sufficient number of deflections were read, accurate to ± 0.01 in, to make it possible to plot the deflected shapes of the slabs. On slab 4D1 the slip at the end of one bar was read by means of a Federal dial.

The device for measuring deflections was suggested by Professor T D Smith of the Civil Engineering staff. A dished piece of metal used on the bottom of furniture legs—with a slight center punch mark to serve as a measuring point, was fastened wherever desired to the surface of the slab with a dab of plaster of paris. A "level rod" was made of a piece of stick, with a pointed nail in the bottom, and having a machinist's scale graduated in hundredths of an inch clamped to one side. An engineer's level, placed far enough away to prevent its being dis-

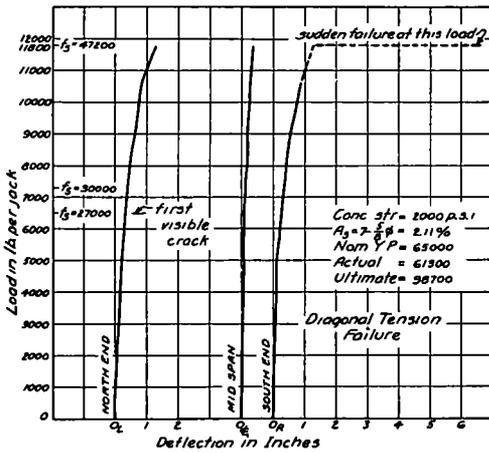


Figure 13
Slab 3 D 1

turbed, was used, and the accuracy was such that subdivisions of the scale could have been estimated, if necessary.

TEST DATA

Deflections were read and observations for cracks were made on each slab following the application of each increment of live load, the deflections due to the weight of the slab itself and to the weight of the jacks and beams between jacks and slab being taken as zero. The end and mid-span deflections of each continuous slab and the midspan deflection of each simple

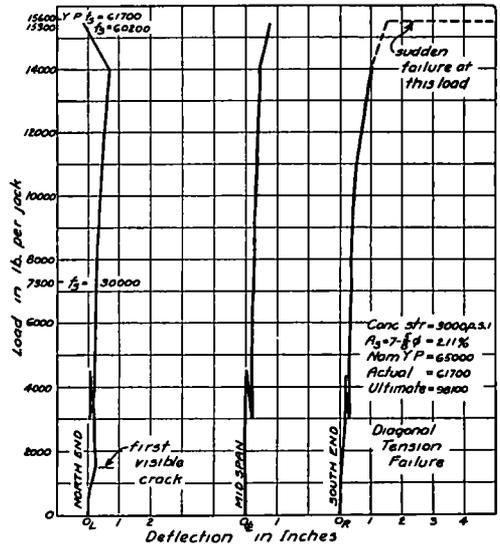


Figure 14
Slab 3 D 2

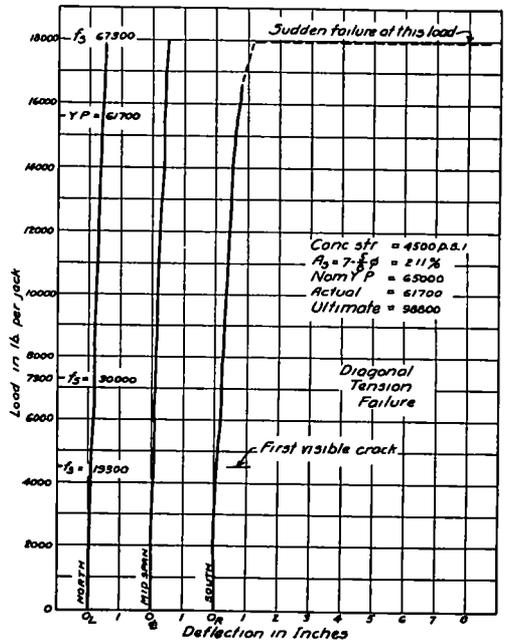


Figure 15
Slab 3 D 3

slab are shown graphically on Figures 9 to 19. All significant features, such as occurrence of cracks, duration of sus-

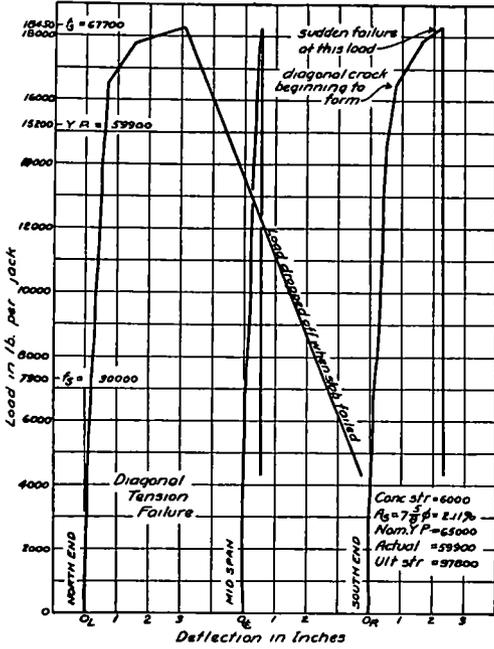


Figure 16
Slab 3 D 4

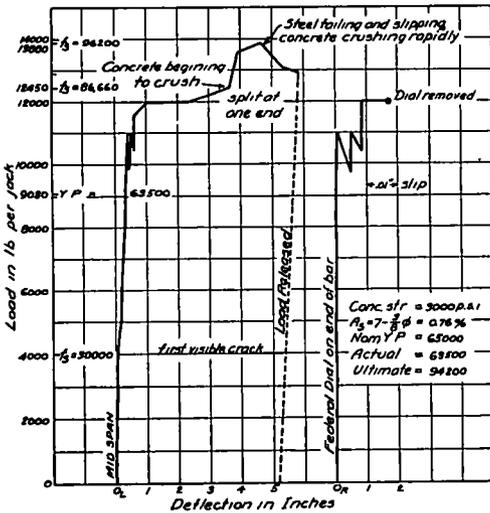


Figure 17
Slab 4 D 1

tained load, release and repetition of load are noted on these figures

The ordinates are in terms of load applied by each jack, hence the effect of

dead load does not appear on the graphs The computed steel stresses, however, which appear at significant points on the graphs, include the effect of dead load For ready reference the kind of concrete,

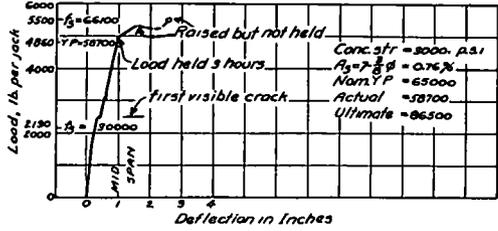


Figure 18 a
Slab 4 D 2

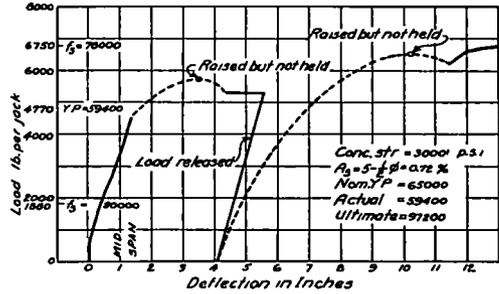


Figure 18 b
Slab 4 D 3

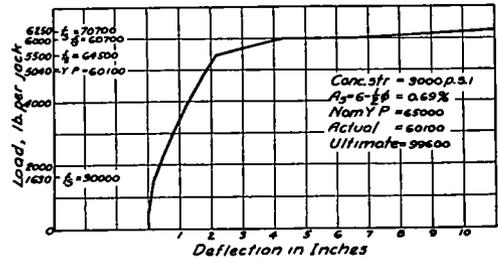


Figure 19
Slab 4 D 4

nominal and actual yield points of the steel and percentage of steel have been repeated In some cases the type of failure has been designated

It should be noted that since the moment at midspan in the continuous slabs was only one half that over the supports, it was impossible to stress the

steel at midspan to the yield point. Except for the influence of slight variations in the depth of slab, the steel stress at midspan, after cracks have appeared on the lower surface, should be one half those indicated at the various significant points on the graphs.

DETECTION OF FIRST CRACKS

Since the principal objectives of the program were directed toward its broader aspects, no refinements of observation were resorted to in the detection of the first crack in any specimen. There are, however, at least five means of detection.

First, direct observation. Even with a magnifying glass and a whitewashed surface this method is usually late in revealing cracks, as was proven long ago by the "water-mark" method of Turneaure and Maurer. Hence on the accompanying graphs reference is made to the first *observed* crack. Water marks were out of question in a program of air-dry specimens.

A third and fourth methods were made possible by the loading apparatus employed. In a quiet laboratory, without the hum of testing machine motor or gears, incipient cracks made themselves heard in a faint though distinct checking or "chicking" noise. With an increase in load these sounds became more frequent, until after the formation of the first few cracks one could, by applying his ear to the slab, hear a continuous chatter which lasted for some time after the load was applied. This indicates that while one or two visible cracks might form at the application of a given load, the process of adjustment within the slabs proceeded for some time. These sounds should not be confused with those which indicate bar slip. The latter are always lusty bumping noises.

The fourth method is even more simple. With equal increments of live load, approximately the same number of strokes of the pump handle were required

to raise the scale beam. Any change in elastic conditions, such as the formation of a crack, at once became evident by a marked increase in the number of strokes required.

The fifth method is a study of the graphs. Usually the first deviation from a regular curve is due to the formation of a crack which soon after becomes visible. For example, (see Fig 10a), the first crack in slab 1D4 was beginning to form over each support at a load of 2000 lb per jack. This method is obviously less reliable the higher the percentage of steel.

DEFLECTIONS

The deflection at the end of the cantilevers in the continuous specimens should be approximately $\frac{1}{3}$ as great as at midspan.

As soon as the elastic condition at the supports is exceeded, however, a conformation of the slab differing from that attained within the elastic range begins to manifest itself. The negative moment at the supports is still that required for complete fixation of that part of the slab between supports, and the positions of the points of contraflexure under increasing load remain unchanged until inelastic conditions occur at midspan. All yielding of the steel or concrete at the support, and all slip of the negative reinforcement on both sides of the support merely permit additional deflection of the overhanging ends of the slab, which are in the condition of cantilevers with deficient restraint (See Fig 3).

In a fully continuous slab, however, yielding over the supports increases the positive moment and causes the points of contraflexure to approach the supports. Hence, even though in the tests of continuous slabs herein reported the loads in every case were raised until inelastic yield over the supports permitted great deflection of the ends, nevertheless deflections at midspan under these loads

were not so great as they would have been in fully continuous slabs. Therefore, except within the elastic range over the supports, it could be very misleading to make deductions concerning deflections in continuous slabs using these tests as a basis.

DISCUSSION OF THE GRAPHS

The failure of the slabs in each group was accompanied by phenomena definitely peculiar to that group. Each will be scrutinized in turn.

The slabs of Group 1D were under-reinforced, although the high yield point of the reinforcement in some of the

steel stress was much higher than the known ultimate strength of the steel. Again, in the case of slab 1D4, the load was raised till the computed steel stress approached the known ultimate, when the test was stopped for fear of accident.

Finally slab 1D3 was loaded to failure after the yield point had been passed, and it was found possible to break the bars in tension. The same procedure was followed in slab 1D5, except that the load was raised until the computed steel stress was equal to the known ultimate strength of the steel, and was then maintained until the bars broke in tension two days later.

TABLE 2

Slab	Y P	Ult Str	Computed f_s		Manner of Failure
			At 1st Vis Crack	At Failure	
1D1	53300	72800	61600	93300	Deflection beyond range of apparatus (2)
1D2	60800	88800	54400	100400	" " (1)
1D3	58200	82800	47500	101300	Bars broke (2)
1D4	72900	115200	64800	105800	(4) (1)
1D5	112000	152000	67000	157000	Bars broke (3)

(1) Load carried straight to maximum

(2) Load carried straight to maximum after passing Y P

(3) Maximum load held 2 days, intermediate loads 3 and 9 days

(4) Load could have been carried further, but was discontinued

slabs increased its effectiveness. Yet consider the data in Table 2 in conjunction with the graphs. Failure was in every case due to yielding of the reinforcement. Before cracks had formed the slabs behaved much like homogeneous members. At the formation of a crack the tension formerly taken by the concrete was thrown into the steel. Where the steel ratio was low, as in the slabs of Group 1D, and where the yield point was not high, as in slab 1D1, first cracking was quickly followed by failure. The computed steel stress in 1D1 at failure, however, was far above the known ultimate strength of the bar. Similarly, in 1D2, the load was raised until the computed

Such phenomena merit serious thought. The writer suggests that it may be possible for a bar to resist a stress, confined to a length of little more than the width of a crack, higher than that resisted by the longer specimens broken in a testing machine. It has been mentioned that the process of readjustment continues for some time, and it is known that bars decrease in diameter before necking begins. So it is hardly too much to suggest that in two days' time the bars of slab 1D5, after continued decrease in diameter and consequent slip, approached the condition of testing machine specimens, and failed at corresponding stresses. The behavior of slab 1D5, therefore,

indicates that the "excess strength" exhibited by under-reinforced specimens loaded quickly to failure, may not be a phenomenon to be depended upon in design

Slab 1D3 was subjected to several repetitions of loads of from 750 to 1500 lb per jack, and it will be noted that in this slab, possibly because of the repetitive load, the first visible crack occurred at a lower computed steel stress than in the other slabs of the group

While the phenomenon of continued deflection was evidenced in many slabs at steel stresses above the yield point, slab 1D5 was twice subjected to sustained loads below the yield point stress, with indications of continued increase in midspan deflection. It will be observed on Figure 10b that such yielding was first noticed at a steel stress far higher than anything so far proposed, yet still below the yield point. The phenomena accompanying the slow application of, or the sustaining of loads is something that will bear much study in all building materials and members

In the slabs of Group 2D the steel ratio was more nearly normal. Here, as in Group 1D, the computed steel stresses developed were all in excess of the yield point, but in the 2D group the ultimate strength of the steel was not attained in any slab. Excessive deflections did not take place till after the yield point of the steel had been passed. Because of the greater, though not unusually high, steel ratio, the total tension and compression at the critical cross section over the support were both greater than in series 1D. Because of this greater total compression and as a result of the transfer to the cantilever of the inelastic phenomena near the support, with its accompanying exaggerated deflection, the concrete at the compression (lower) surface of all the slabs of this series was crushed at earlier loads than might have been expected. In no case did the con-

crete stress, computed by the straight line formula, approach the cylinder strength of the concrete. While these compression failures were secondary, they effectually prevented the application of loads high enough to cause steel failure. Had a different method of obtaining continuity been employed, as, for example, making the slab two panels long, the points of contraflexure near the middle supports would have been free to shift to conform with changes in elastic conditions, and it is possible that somewhat higher loads could have been carried

One significant point, clearly brought out by comparing the strength of slab 2D1 with that of slab 2D1X is that in secondary compression failures the strength of the concrete is relatively unimportant

The heavily reinforced 3D Group, in which the strength of the concrete varied from 2000 to 6000 lb per sq in while the nominal yield point of the steel remained constant, furnishes a remarkably interesting set of comparisons, and makes imperative a study of concrete stresses when compression failure occurs. All specimens failed nominally by diagonal tension, with cleavage along the plane of the negative reinforcement. In slabs 3D1, 3D2 and 3D3 the concrete stress, computed by the straight line formula, exceeded the cylinder strength of the concrete in a decreasing ratio, while in slab 3D4 it was not much below the cylinder strength

In slab 3D1 failure occurred before the yield point of the steel had been reached, while in slab 3D2 yield point and diagonal tension failure were reached practically simultaneously. In slabs 3D3 and 3D4 the yield point stress was exceeded before failure occurred, the excess being greater in slab 3D4

To make these phenomena understandable it will be necessary merely to resort to the simple theory of the second de-

gree parabolic distribution of stress, announced in 1906 by Dr A N Talbot, published in University of Illinois Bulletin No. 4, and developed in "Principles of Reinforced Concrete Construction" by Turneure and Maurer. This is summarized for reference on Figure 20 For example, if the stress at the extreme fiber, computed by the straight line formula, is 1.4 times the cylinder strength, this stress, by the parabolic formula, is determined at $0.91f_c'$. The value of q is 70 and, if accuracy is required in the determination of steel stress, this value may be used in finding j by means of the formula in "Turneure and Maurer." Suffice it to say that the new steel stresses will

Corresponding to this value on the straight line we have, on the parabola, the value 0.995, and

$f_c = 995 \times 2000 = 1990$ lb per sq in which checks the fact that the concrete had reached the point of failure. If one cared to go further he could find on the curve that $q = 92$ and, by means of the formulas mentioned, that $k = .60$ and $j = .78$. The revised steel stress would be 49800 instead of 47200 lb per sq in.

With this in mind, Table 3 and the manner of failure it portrays will be more intelligible.

TABLE 3

Slab No	Kind of Cone	Y P of Steel	Computed f_c		Unit Bond ¹	Unit Shear ²	
			St L	Par			
3D1	2000	61300	3690	1990	47200	369	141 ³
3D2	3000	61700	4680	2850	60200	467	183 ³
3D3	4500	61700	5260	3690	67300	524	210 ⁴
3D4	6000	59900	5290	4200	67700	526	216 ⁴

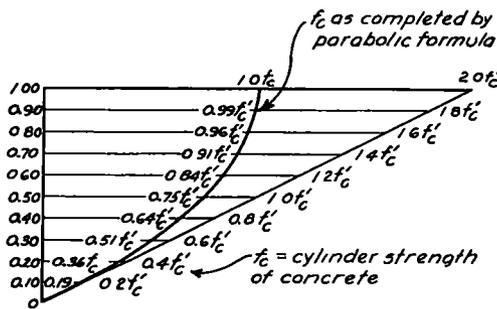


Figure 20 Comparison of stresses as determined by straight line and parabolic formulas.

rarely be 5 per cent greater than those given by the straight line formula.

Returning again to the slab tests let us examine slab 3D1 at the failure load, 11800 lb per jack. We have.

$$\begin{aligned} \text{Moment due to slab and jacks } & 11000 \text{ in lb} \\ \text{" " " Load = } & \\ 11800 \times 20 \times 25 & = 239000 \text{ in lb} \\ \text{Total Mom} & = 250000 \text{ in lb} \end{aligned}$$

By the straight line formula:

$$f_c = \frac{250000 \times 2}{82 \times 54 \times 34 \times 3^3} = 3690 \text{ lb per sq in}$$

$$\text{Now } \frac{3690}{2000} = 1.845$$

¹ Computed by bond formula $u = \frac{V}{\Sigma o_j d}$

² Computed by shear formula $v = \frac{V}{bjd}$

³ Failure by diagonal tension Compression high

⁴ Failure by diagonal tension Tension high

Figure 4, showing the failure of slab 3D2, is typical of the whole group. The diagonal tension failure is scarcely to be wondered at when the high bond stress is considered together with the high unit shear in a member not reinforced to resist shear. The failure was always sudden, at the maximum load reached. That bond had a good deal to do with the failure is confirmed by comparing these failures with the diagonal tension failures reported by Professor Lyse in Journal A. C. I. Sept-Oct 1936. The cantilever ends of his continuous slabs were just long enough to engage the outer load, and in them diagonal tension

failure occurred outside the supports. In our tests the cantilevers were longer, and diagonal tension failure occurred inside the supports. It is therefore obvious that in a continuous span of 8 ft, $\frac{3}{8}\phi$ bars are of so large diameter relative to the distance from point of contraflexure to support that bond failure is almost inevitable and will contribute to, if not precipitate, failures of other kinds. It would be interesting to test a duplicate

had slipped as much as $\frac{3}{8}$ in. around the bends. Its condition at this load is shown in Figure 5. There was no evidence of diagonal tension failure in any specimen, and only in 4D1 was there any measurable slip. Table 4 presents the essential data in predigested form.

From Table 4 many interesting comparisons may be made, as shown in Table 5.

Slab 4D2, which reached the limit in

TABLE 4

Slab	Span	Thick	Conc	A _s	p	Y P	Computed f _c		Computed f _s	Unit Bond ⁴	Unit ⁴ Shear
							St L	Par			
4D1	5'-0"	4"	3000	7 ϕ	76%	63500	5460	2910	76800	475	115 ¹
4D2	8'-0"	4"	3000	7 $\frac{1}{2}\phi$	76%	58700	6160	3000	86600	536	130 ²
4D3	12'-0"	5"	3000	5 $\frac{1}{2}\phi$	72%	59400	4700	2820	66100	254	62
4D4	16'-0"	6"	3000	6 $\frac{1}{2}\phi$	69%	60100	1675		78000	260	60
							1575		70700	169	47

¹ At first end slip of bar

² At jack load of 12450, conc beginning to crush

³ Computed by bond formula $u = \frac{V}{\sum o_j d}$

⁴ Computed by shear formula $v = \frac{V}{bjd}$

TABLE 5

Slab	$\frac{L}{t}$	Mom ¹ Ratio	at Y P	Δ/L	$\frac{\Delta}{L}$ Ratio	Ult Str Conc	Y P of Steel	Bond
4D1	15	96.5%	32"	0017	1.0	Developed	Passed	Failed
4D2	24	89.4	90	0063	3.7	About developed	"	About limit
4D3	29	85.6	1.50	0156	9.2	Not developed	"	" "
4D4	32	75.7	1.86	0310	18.2	" "	"	Not high

¹ Ratio of L L mom to total mom

slab in which the same area of steel was used in smaller sizes. Even at the risk of becoming tedious, it should again be pointed out that in a continuous slab high bond and tensile stresses occur at the same place, which is not the case in a simply supported slab.

The slabs of Group 4D were all simply supported and were of various spans. Here again it was possible to produce compression failure in one of the slabs, 4D1, but only after the hooks on the bars

concrete and bond strengths simultaneously without developing an excessively high unit shear is perhaps the best balanced slab. A little larger steel ratio would have been beneficial.

The deflections of slabs 4D3 and 4D4 are much greater than would be anticipated, particularly that of slab 4D4. Whether the moment of inertia be assumed roughly as $\frac{bt^2}{12}$ or determined "exactly" from the transformed area of

the cross section, the observed deflections are much greater than the computed deflections. From this we are led to believe that when the ratio of span to thickness in simple slabs becomes much over 24, the designer is not only penalized to a certain extent in the moment ratio, but is penalized heavily in the deflection of his slab. These tests, though so few in number, indicate that there is a limit to effective span-thickness ratios.

SUMMARY

Briefly, in the 1935-36 program of slab tests conducted at the University of Delaware the following phenomena were noted:

(1) With low steel ratios the yield point of the steel may be passed with the formation of the first crack. The minimum steel ratio advisable is that which will make the cracked reinforced slab as strong as the uncracked unreinforced slab. This steel ratio will obviously be greater in slabs and rectangular beams than in T-beams.

(2) Cantilevers were found to be subject to undue deflection, after cracking had taken place, because of the transfer to the cantilever of all the inelastic phenomena near the support.

(3) In all specimens, except in slabs 3D1 and 3D2 where failure was due to other causes, the Y P of the steel was developed, at proportionate moments, even when it was as great as 112,000 lb per sq in.

(4) The behavior of slabs reinforced with different effective steel ratios was as follows:

(a) When low percentages of steel were used the full ultimate strength of the steel was developed at loads not greatly differing from relative actual yield points or relative ultimate strengths.

(b) With intermediate steel ratios, such as commonly used in design, secondary negative compression failure took place, the strength of the concrete having little influence on the load carried.

(c) High steel ratios prevented sec-

ondary compression failure, and greatly increased the strength of the slab, until bond or diagonal tension failure made further increase impossible.

(5) The amount of steel developed by the concrete in the latter case was high. In compression due to flexure 4500 lb concrete more than developed 2.11 per cent of steel of 60,000 Y P, while 3000 lb concrete barely fell short.

(6) Apparently the effective steel ratio has more to do with the strength of a concrete slab than has the compressive strength of the concrete, unless the concrete is abnormally weak, and the region of negative moment is one of especial concern.

(7) It is possible that at a crack the steel stress may for a short time exceed the ultimate strength of tension coupons.

(8) Holding the load which produced a calculated stress equal to the ultimate strength of steel tension coupons eventually ruptured the bars, thus making it questionable whether in design any dependence may be placed on the "excess strength."

(9) At a computed steel stress of 30,000 lb. per sq in all cracks were hair cracks.

(10) Repetition of a load caused the concrete to crack much earlier than otherwise.

(11) While holding the higher loads produced rapid progressive deflections, holding those which produced steel stresses below the yield point of high strength steel also caused deflections not to be ignored. See Figure 10, slab 1D5.

(12) The ratio of span length to slab thickness is limited both by reasons of deflection and economy.

(13) In any curve showing the effect of sustained load, the points of sag are the significant ones.

(14) Under static load, even under extreme conditions of stress and deflection, the behavior of Rail steel could not be distinguished from that of Billet steel.