1939

Closing discussion. High yield point steel as tension reinforcement in beams.

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183.1

Vol. 36 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400, SECOND BOULEVARD, DETROIT, MICHIGAN *SEPTEMBER 1939*

High Yield-Point Steel as Tension Reinforcement in Beams*

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SYNOPSIS

Results of tests of 32 rectangular concrete beams reinforced with four different types of high yield-point steels are presented in this report. The beams had an effective depth of 12 in., a width of 12 in. and a distance center-to-center of supports of 9 ft. The four types of steel used were: (1) hard grade steel, (2) nickel steel (one beam only) (3) square twisted bars, and (4) "twin-twisted and stretched" bars.

Results show that when a concrete beam is reinforced against diagonal tension failure the strength is determined by the total yield strength of the steel (steel area times yield-point stress) and not by the type of steel.

FOREWORD AND ACKNOWLEDGMENT

This investigation, sponsored by the Concrete Reinforcing Steel Institute, was started in 1937 under the direction of Inge Lyse, formerly Research Professor of Engineering Materials at Lehigh University and now Professor of Reinforced Concrete and Solid Bridges at the Norges Tekniske Hoiskole at Trondheim, Norway. The investigation was a regular research project of the Fritz Engineering Laboratory, which is under the administrative direction of Prof. Hale Sutherland, Head of the Department of Civil Engineering. Howard Godfrey, Engineer of Tests at the Fritz Engineering Laboratory, contributed continuous assistance and advice during the entire investigation.

*Received by the Institute, March 27, 1939.

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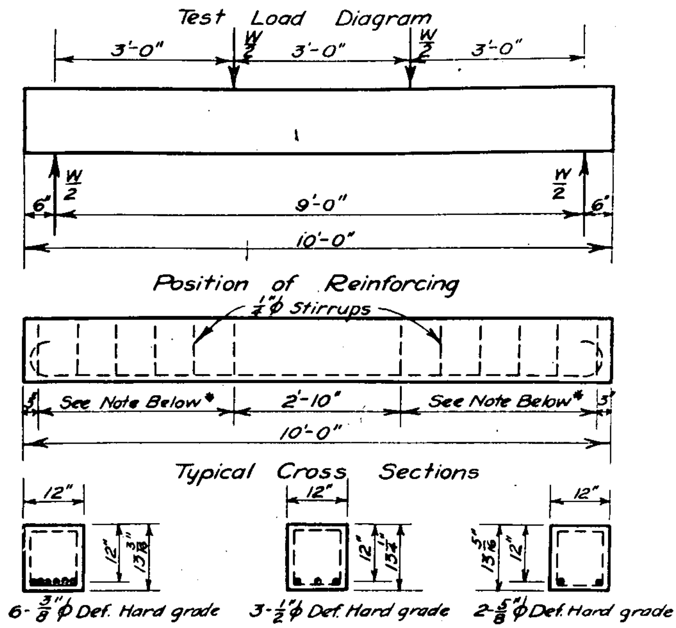


STIRRUPS

Beam Number	Number of Stirrups	Beam Number	Number* of Stirrups	Beam Number	Number* of Stirrups
A-1	6	C-3	14	N-1	14
A-2	6	C-4	15	I-1	6
A-3	6	T-1	6	IS-1	6
B-1	6	T-2	6	IS-2	11
B-2	6	T-3	6	IS-3	14
B-3	6	ST-1	11	IS-4	15
C-1	6	ST-2	14		
C-2	11	ST-3	14		

*NOTE: Number of ¼φ def. stirrups evenly spaced at each end of the beam.

FIG. 1—LOAD DIAGRAM AND LOCATION OF STEEL

INTRODUCTION

Purpose—The purpose of this investigation is to study the behavior of various types of high yield-point steels as tension reinforcing in concrete beams.

The question of adopting increased allowable unit stresses for high yield-point strength steel reinforcing is of current interest among designing engineers. In some localities higher stresses have been allowed for special types of steels in which the yield point has been raised by simultaneously stretching and twisting two round bars together.

A previous investigation considered principally "twin-twisted and stretched" bars in comparison with structural grade carbon steel [1,2]. The present program has been designed to coincide with the previous tests in regard to dimensions of specimens and strength of concrete so that the data from both sources would be directly comparable.

TEST PROGRAM

Thirty-two beams were made for this program. Several sizes of each type of bar were used except in the case of the nickel steel. The variables include the type of steel, the percentage of steel, and the size of steel. Table 1 shows the type, size, and amount of reinforcing used in each beam. Modulus tests on concrete and steel were also determined. The general dimensions of the test beams and loading arrangement are shown in Fig. 1. A photograph of a typical beam in the testing machine prior to loading is shown in Fig. 2.

Steel—Physical properties of the steels used are given in Table 2. The various types of bars are shown in Fig. 3. The first bar on the left is the ⅝-in. nickel steel bar, the next four bars from the left are the hard grade bars, the next three are the square twisted bars, and the last five are the various sizes of "cold-twisted and stretched" bars The hard grade deformed bars were furnished by the Truscon Steel Co. Youngstown, Ohio. The yield point was noted by the "drop of the beam" method. Nickel steel for one beam was furnished by the International Nickel Co. of Bayonne, New Jersey. The yield point was determined by the A. S. T. M. offset method of 0.2 per cent elongation on both the nickel and square twisted steel. The square twisted steel was donated by the Bethlehem Steel Co., of Bethlehem, Pa. The No. 1 and No. 2 Isteg bars "twin-twisted and stretched" were purchased.

Other ⅜-in.$\phi\phi$, ½-in.$\phi\phi$, and ⅝-in.$\phi\phi$ "twin-twisted and stretched" bars were donated by the Bethlehem Steel Co. and were manufactured by methods identical with the bars used in the previous investigation[1]. Coupons from all "twin-twisted and stretched" bars were cut in 2½ ft. lengths and welded for 2-in. on each end to enable the bars to work together. The yield point was obtained by the A. S. T. M. offset method.

[1]"Isteg Steel for Concrete Reinforcement" by D. B. Steinman, JOURNAL, Amer. Concrete Inst., Nov. 1935; Proceedings Vol. 32, p. 183.

[2]"The Modular Ratio—A New Method of Design Omitting "*m*", *Concrete and Constructional Engineering*. Mar. 1937. p. 189. K. Hajnal-Konyi.

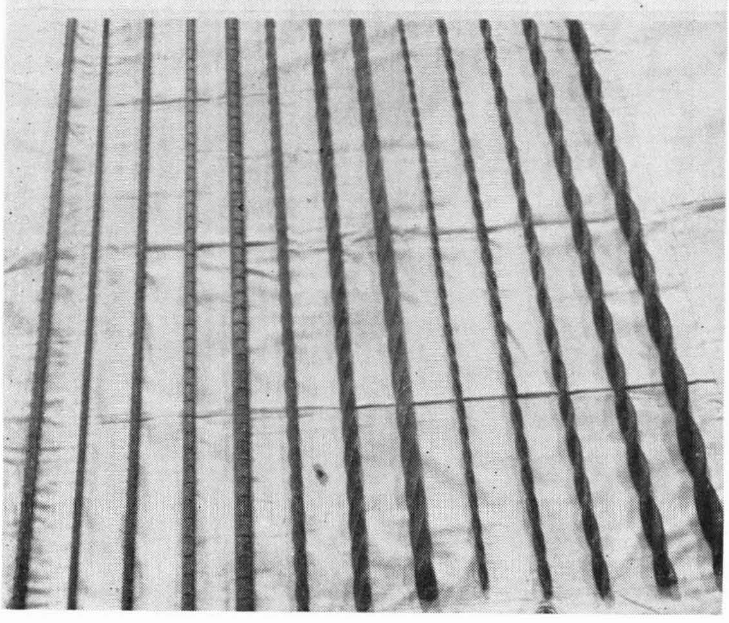
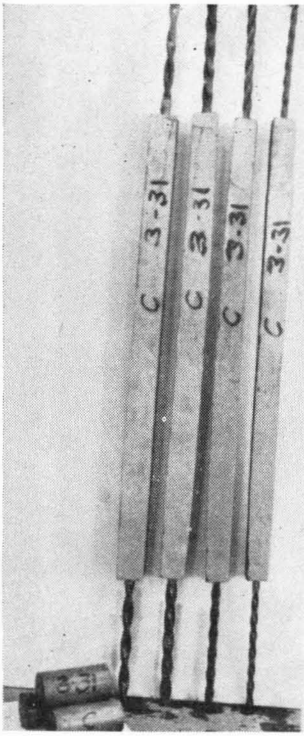


FIG. 2 (TOP)—TYPICAL BEAM IN TESTING MACHINE

FIG. 3—TYPES OF BARS TESTED



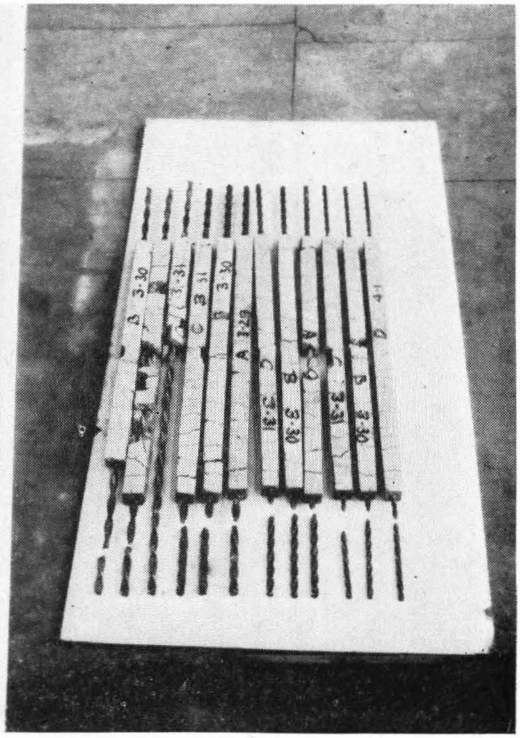


Fig. 4 (left)—Four specimens before testing

Fig. 5 (right)—Bars after removal from testing machine

A series of tests was made to determine the effect of embedment on square twisted and "twin-twisted and stretched" bars. Three test specimens were made for each of the following bar sizes: $\frac{1}{2}$-in. square twisted, $\frac{5}{8}$-in. square twisted, $\frac{3}{4}$-in. square twisted, and $\frac{1}{2}$-in$\phi\phi$ "cold twisted and stretched." These bars were embedded in the center of a square concrete block 42 in. long with a $\frac{3}{4}$-in. coverage at the nearest face. Fig. 4 shows four of the specimens before testing. The "twin-twisted and stretched" bars were welded for a few inches on each end to keep them working uniformly in the grips of the machine The deflection in the 40-in. gage length was measured by two Ames dials reading to the nearest 1/1000 in.

Fig. 5 shows the bars just after removal from the testing machine. The concrete spalled off to a greater degree in the "twin-twisted and stretched" bars than in the square twisted bars, because longitudinal cracks developed in these bars in addition to the transverse cracks.

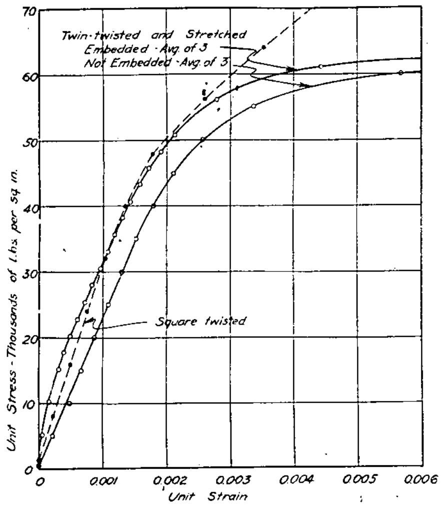


FIG. 6—STRESS-STRAIN DIAGRAMS FOR TWISTED REINFORCING STEEL

Observations showed that the modulus of square twisted bars was unchanged by embedment. The apparent modulus of the "twin-twisted and stretched" bars was raised in the initial range before failure of concrete in tension but at stresses greater than 15,000 p.s.i. it became practically the same as for the unembedded condition. In the working stress range the modulus of the "twin - twisted and stretched" bars was approximately 22,000,000 p.s.i. in both the unembedded and embedded tests, as can be determined by Fig. 6.

Concrete—The concrete was designed for 3300 p.s.i. at 28 days to correspond with the previous investigation[1].

A cement-water ratio by weight of 1.28 was used with 300 lb. of water per cubic yd. of concrete to give the desired workability. Pit sand from northern New Jersey was used for fine aggregate. The ⅜-in. and ¾-in. crushed limestone rock used as coarse aggregate was donated by the Bethlehem Steel Corp., Bethlemem, Pa. The cement was donated by the Lehigh Portland Cement Co. The proportion of sand to coarse aggregate was established at 1:2 and the proportion of ⅜-in. coarse to ¾-in. coarse was made 1:2 also.

Ten control cylinders were made for each pair of beams for the first 20 beams. For each of the last 12 beams five control cylinders were made and no strain readings were recorded. The average 28-day compressive strength of the cylinders for the first 20 beams is 3190 p.s.i., for the last 12 beams 3220 p.s.i.

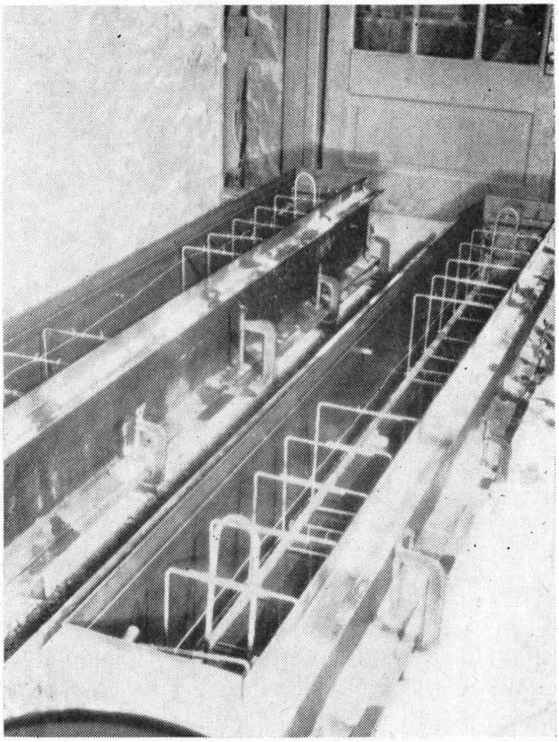


FIG. 7—REINFORCING STEEL IN FORMS

Beams—The beams had an effective depth of 12-in., a width of 12 in., and an overall length of 10 ft. Supports were nine feet center-to-center and third-point loading was used as shown in Fig. 1. The center of gravity of the steel was adjusted to exactly 12 in. by using various screeds which would give this desired depth. The steel was wired together before it was placed in the steel forms which are shown in Fig. 7.

In the first 20 beams an 8-in. stirrup spacing was used. Twelve of these beams were reinforced with hard grade deformed bars, 6 with square twisted bars, and 2 with "twin-twisted and stretched" bars. For beams with nearly equal steel areas diagonal tension failure resulted in 5 out of 6 beams with square twisted bars, both of the beams with "twin-twisted and stretched" bars, but in only 3 out of 6 beams with straight bars. This indicated a slight tendency toward diagonal failure in the case of beams with the square twisted bars and "twin-twisted and stretched" bars. The last 12 beams were designed to eliminate

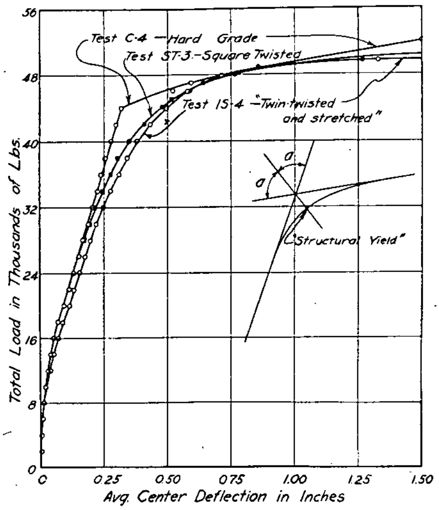


FIG. 8—TYPICAL LOAD DEFLECTION DIAGRAMS FOR BEAMS WITH DIFFERENT TYPES OF REINFORCING

diagonal tension failure by use of additional stirrups. Fig. 1 indicates the stirrups used in the various beams. Four of these beams were reinforced longitudinally with hard grade deformed bars, three with square twisted, four with "twin-twisted and stretched," and one beam was reinforced with nickel steel. The stirrups used in all cases were intermediate grade ¼-in. diameter deformed bars with the bamboo or diamond deformations.

The concrete was mixed in 2¼ cu. ft. batches. Each batch was given a three-minute mix. Steel plugs were cast in the compression side of each beam in order to measure the compression strains in the concrete.

At the age of one day the forms were stripped and the beams were placed in the moist room until the age of 28 days at which time they were tested. The specimens were kept damp until they were placed in the testing machine.

Strain readings were taken on both the steel and concrete with a Whittemore strain gage measuring strains to the nearest 1/10,000 in. over a 10-in. gage length. Huggenberger readings were also made on some of the first beams tested but were discontinued because of difficulty encountered in attaching them to the curved surface of a reinforcing bar.

TABLE 1—BEAM DATA

Beam No.	Number and Size of Bars	Type of Bar	Area of Steel sq. in.	Yield Point Stress of Steel p.s.i.	Total Yield Strength of Steel in Beams lbs.	Structural Yield Point Load lbs.	Ultimate Load lbs.	Type of Failure *
C-1	3 ⅜″ φ def.	Hard Grade	0.324	62 200	20 160	12 500	17 950	T.
C-2	3 ½″ φ def.	"	0.560	59 200	33 000	19 900	27 750	T.
C-3	3 ⅝″ φ def.	"	0.890	60 300	53 610	32 400	39 600	T.
C-4	4 ⅝″ φ def.	"	1.184	60 300	71 480	44 650	51 800	T.
A-1-I	6 ⅜″ φ def.	"	0.650	62 200	40 300	26 650	30 600	T.
A-1-II	6 ⅜″ φ def.	"	0.650	62 200	40 300	25 500	30 900	T.
A-2-I	3 ½″ φ def.	"	0.560	59 200	32 500	21 150	28 650	T.
A-2-II	3 ½″ φ def.	"	0.560	59 200	32 500	21 100	28 400	T.
A-3-I	2 ⅝″ φ def.	"	0.590	60 300	35 200	23 300	28 300	T.
A-3-II	2 ⅝″ φ def.	"	0.590	60 300	35 200	24 400	28 200	T.
B-1-I	6 ½″ φ def.	"	1.120	59 200	65 000	41 000	45 200	D. T.
B-1-II	6 ½″ φ def.	"	1.120	59 200	65 000	42 200	45 300	T.
B-2-I	4 ⅝″ φ def.	"	1.184	60 300	70 900	42 100	44 500	T.
B-2-II	4 ⅝″ φ def.	"	1.184	60 300	70 900	43 800	45 150	D. T.
B-3-I	2 ⅞″ φ def.	"	1.193	63 200	75 300	45 050	52 800	T.
B-3-II	2 ⅞″ φ def.	"	1.193	63 200	75 300	45 800	52 850	D. T.
I-1-I	3 ½″ φφ Isteg	Twin Tw. and Stretched	1.180	58 600	69 200	43 800	46 000	D. T.
I-1-II	3 ½″ φφ Isteg	"	1.180	58 600	69 200	42 300	45 500	D. T.
T-1-I	5 ½″ Sq. tw.	Sq. Twisted	1.250	58 400	73 000	43 500	54 150	D. T.
T-1-II	5 ½″ Sq. tw.	"	1.250	58 400	73 000	45 200	54 250	T.
T-2-I	3 ⅝″ Sq. tw.	"	1.165	61 800	72 300	44 300	47 100	D. T.
T-2-II	3 ⅝″ Sq. tw.	"	1.165	61 800	72 300	43 300	45 700	D. T.
T-3-I	2 ¾″ Sq. tw.	"	1.125	64 400	72 100	40 900	42 800	D. T.
T-3-II	2 ¾″ Sq. tw.	"	1.125	64 400	72 100	42 100	45 500	D. T.
ST-1	2 ½″ Sq. tw.	"	0.500	64 500	32 250	18 200	23 550	T.
ST-2	2 ⅝″ Sq. tw.	"	0.780	58 200	45 400	28 900	34 800	T.
ST-3	2 ¾″ Sq. tw.	"	1.125	61 000	68 300	42 400	49 400	T.
N-1	2 ⅝″ φ def.	Nickel	0.615	83 500	50 200	36 700	42 550	T.
IS-1	3 No. 1 Isteg	Twin Tw. and Stretched	0.244	67 800	16 520	13 200	14 700	T.
IS-2	4 No. 2 Isteg	"	0.582	68 000	39 700	27 600	31 150	T.
IS-3	3 ⅜″ φφ Isteg	"	0.661	57 200	37 800	26 500	30 300	T.
IS-4	2 ⅝″ φφ Isteg	"	1.230	54 000	66 400	46 200	49 600	T.

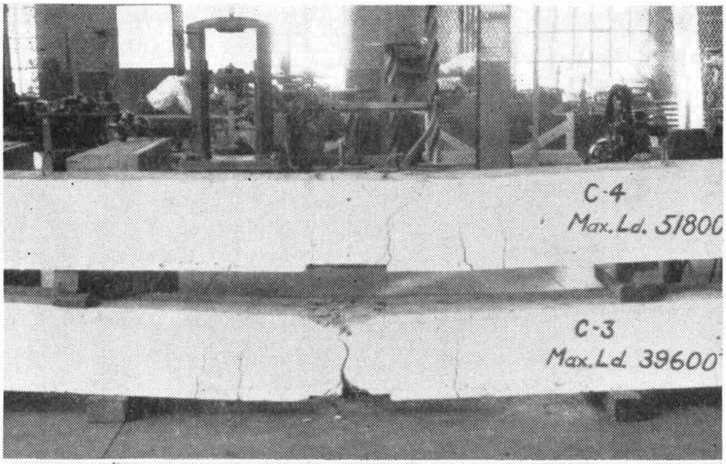
*T. = tension failure in steel.
D. T. = diagonal tension failure.

Deflections were read on both sides at the center of the beams by means of Ames dials reading to the nearest 1/1000 in. Typical beam deflection curves for each type of reinforcing are shown in Fig. 8.

TEST RESULTS

Tests of Materials—Results of tests of materials have been given in the preceding section and in Table 1.

Typical Tests of Beams—The load-deflection curves in Fig. 8 depict the "load-history" of the beams during three typical tests, giving a graphical picture of all stages of failure. The first break in the curve occurs at loads between 4000 and 12,000 lb. at which time the concrete fails in tension. Cracks show up on the tension side of the beam immediately after this failure and these progress in size and number as the load increases. It should be understood that these cracks are of sufficient size to be plainly visible and are not hair line cracks which are made visible only by soaking in water or through other artificial means. The curve then runs uniformly until the load at which the steel begins to yield. At this point the number of cracks depends



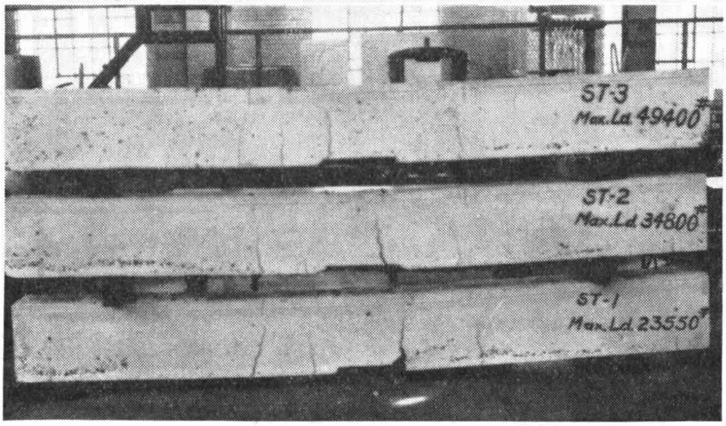


FIG. 9, 10—TYPICAL CONDITION OF BEAMS AFTER THE ULTIMATE LOAD HAD BEEN REACHED FOR BEAMS REINFORCED WITH HARD GRADE DEFORMED BARS AND SQUARE TWISTED BARS, RESPECTIVELY

upon the amount of reinforcing, and for any given number their size depends upon the deflection of the beam. The number of cracks varied from 4 in beams with a low percentage of steel to 18 in the beams with the high percentages. Fig. 9, 10, and 11 show the typical condition of the beams after the ultimate load had been reached for beams reinforced with hard grade deformed bars, square twisted bars, and "twin-twisted and stretched" bars respectively. The upper break in the load-deflection curve will be regarded as the limit of structural usefulness or "structural yield point." The concrete begins to crush shortly after passing the "structural yield point" and the

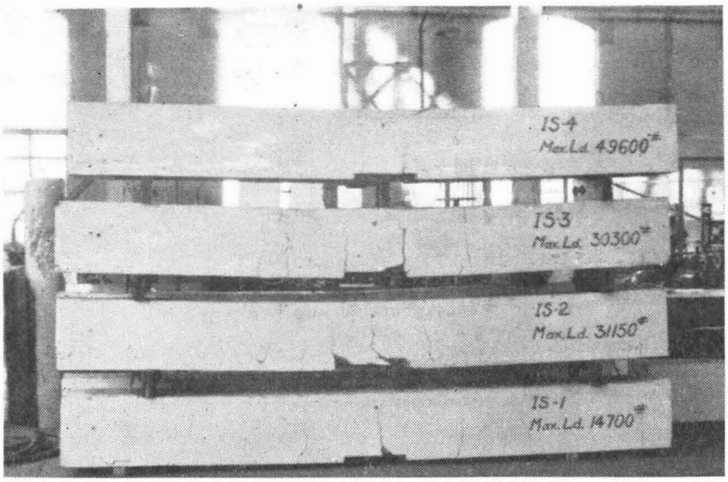
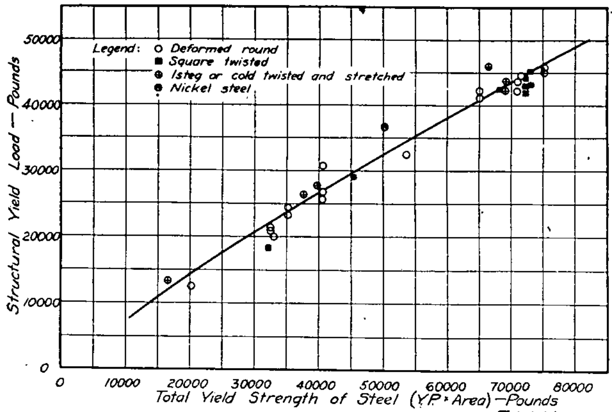


FIG. 11—TYPICAL CONDITION AFTER ULTIMATE LOAD, OF BEAMS REINFORCED WITH "TWIN-TWISTED AND STRETCHED" BARS

TABLE 2—PHYSICAL PROPERTIES OF THE STEEL

Steel	Used in Beams	Method of Obtaining the High Yield Point	No. of Tensile Tests	Yield Point p.s.i.	Ultimate p.s.i.	% Red in Area	% Elong. at 2"	% Elong. at 8"
3/8" ϕ H.Y.P. def.	A-1-I, A-1-II, C-1	High Carbon	12	62 200	92 800	53.8	26.5	17.5
1/2" ϕ H.Y.P. def.	A-2-I, A-2-II, B-1-I, B-1-II, C-2	"	18	59 200	95 100	51.2	38.5	18.3
5/8" ϕ H.Y.P. def.	A-3-I, A-3-II, B-2-I, B-2-II, C-3, C-4	"	12	60 300	93 100	48.4	30.0	18.4
7/8" ϕ H.Y.P. def.	B-3-I, B-3-II	"	4	63 200	106 800	33.9	27.0	15.8
5/8" ϕ H.Y.P. def.	N-1	Nickel	2	83 500	126 900		17.5	8.4
1/2" Sq. twisted	T-1-I, T-1-II	Cold Twisting	10	58 400	70 000	27.2	12.0	5.1
1/2" Sq. twisted	ST-1	"	2	64 500	71 300	29.8	12.5	5.0
5/8" Sq. twisted	T-2-I, T-2-II	"	6	61 800	72 800	31.3	15.5	6.3
5/8" Sq. twisted	ST-2	"	2	58 200	70 000	34.9	16.0	6.5
3/4" Sq. twisted	ST-3	"	2	61 000	70 300	37.7	22.5	9.1
3/4" Sq. twisted	T-3-I, T-3-II	"	4	64 400	75 100	36.5	19.5	7.5
No. 1—Isteg	IS-1	Cold stretching and twisting	3	67 800	85 400	47.2	8.0	2.9
No. 2—Isteg	IS-2	"	4	68 000	79 500	37.4	18.3	
3/8" ϕϕ pl. rd. tw.	IS-3	"	3	57 200	68 800	25.9	12.0	5.4
1/2" ϕϕ pl. rd. tw.	I-1-I, I-1-II	"	3	58 600	72 200	52.8	18.0	6.3
5/8" ϕϕ pl. rd. tw.	IS-4	"	2	54 000	65 800	34.6	20.0	9.1



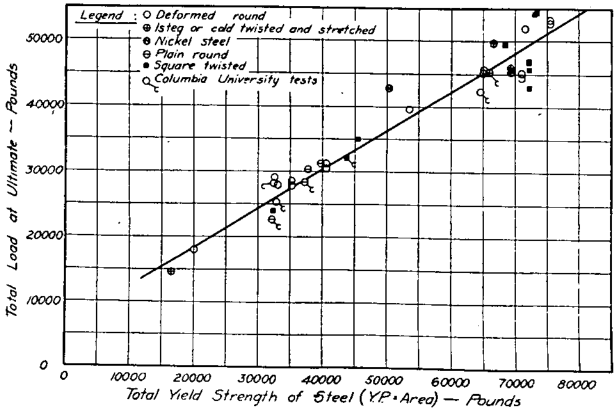


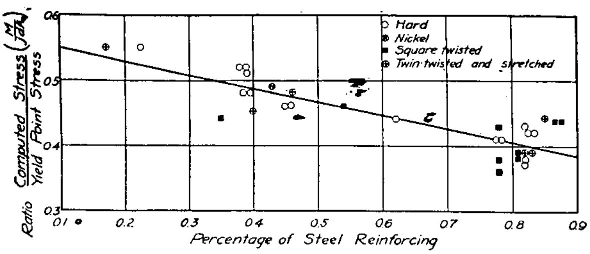
FIG. 12—RELATION BETWEEN STRUCTURAL YIELD AND TOTAL YIELD STRENGTH OF REINFORCING STEEL

FIG. 13—RELATION BETWEEN ULTIMATE STRENGTH OF BEAMS AND TOTAL YIELD STRENGTH OF REINFORCING STEEL

ultimate strength of the beam is quickly reached. The "structural yield point" was arbitrarily determined by the graphical construction shown on the curves in Fig. 8. The construction consisted in bisecting the angle formed by the intersecting extensions of the straight portions of the curve below and above the region of sharp curvature. This method is particularly adapted to the load deflection diagrams corresponding to these beam tests.

SUMMARY OF BEAM TESTS

Fig. 12 presents graphically the relation between total yield-strength of the steel and the structural yield point of the beams. This relation



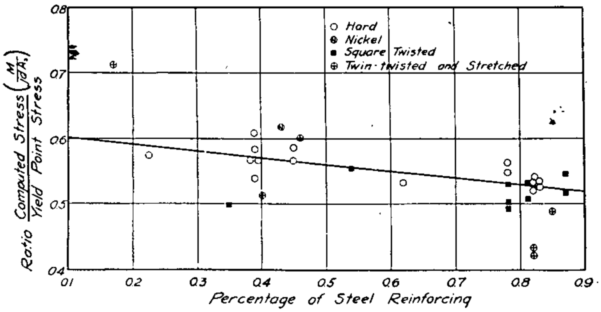


FIG. 14—RATIO OF COMPUTED STRESS AT A BEAM LOAD OF ONE-THIRD ULTIMATE BEAM STRENGTH TO YIELD POINT STRESS OF REINFORCING STEEL

FIG. 15—RATIO OF COMPUTED STRESSES AT BEAM LOAD OF ONE-HALF STRUCTURAL YIELD STRENGTH TO YIELD POINT STRESS OF REINFORCING STEEL

is seen to be nearly linear and is independent of the type of reinforcing steel used.

Fig. 13 shows the relation between total yield-strength of the steel and ultimate strength of the beams. The Columbia tests are included in this diagram and a close agreement is noted with the Lehigh tests. The ultimate strength of the beams is also proportional to the total yield-strength of the steel.

Fig. 12 and 13 show that both the structural yield and ultimate strength of a reinforced concrete beam depend primarily on the total yield-point strength of the steel regardless of the type of bar or manner by which the high yield point is obtained.

Design Loads—Although no definite recommendations will be made in this report as to proper working stresses the test data will be compared at loads of one-third the ultimate and one-half the "structural yield point." Conservative practice would allow the use of the minimum of these two values as a design load. In every beam of the

32 tested the load at one-third the ultimate was smaller than at one-half the "structural yield point." This result was made probable because the structural yield of the beams was always closely followed by ultimate failure.

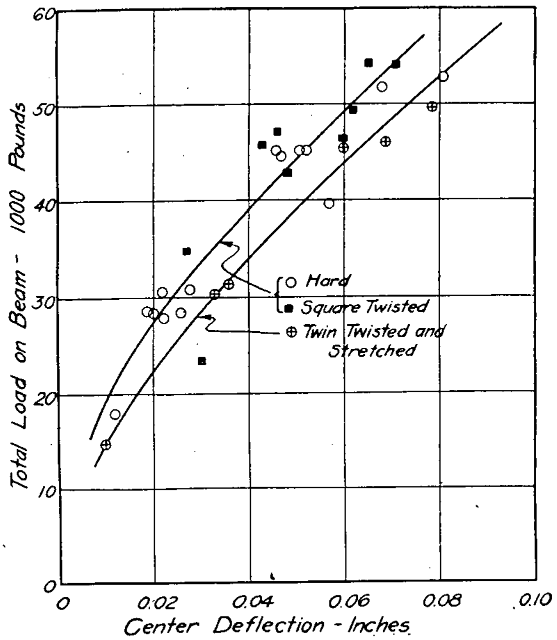


FIG. 16—CENTER DEFLECTION OF BEAMS AT ONE-THIRD ULTIMATE STRENGTH

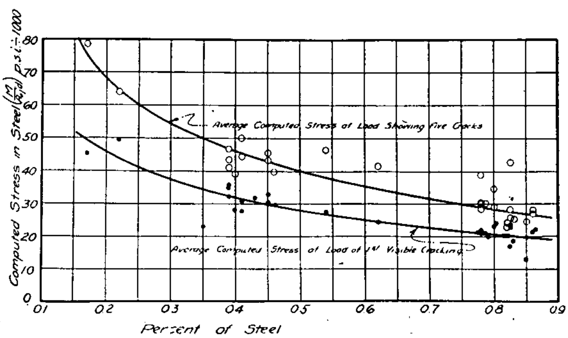


FIG. 17—CURVE SHOWING THE COMPUTED STRESS IN THE STEEL AT FIRST CRACKING AND THE APPEARANCE OF FIVE CRACKS FOR VARIOUS PERCENTAGES OF STEEL

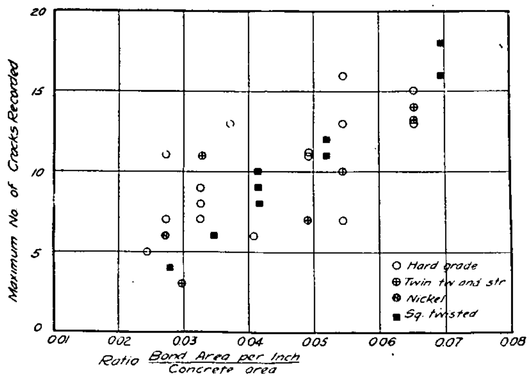


FIG. 18—EFFECT OF BOND ON TOTAL NUMBER OF CRACKS AT ULTIMATE LOAD

If increased stresses are to be allowed for high yield strength steels the allowable working stresses will probably be specified at some percentage of the yield-point strength. Fig. 14 and 15 present the ratio of calculated stress to yield stress of the steel at one-third the ultimate and one-half the structural yield point, respectively. The stress calculation is based on the usual straight-line stress-strain assumption with a value $n = 10$ assumed for straight and square twisted bars and $n = 7.5$ assumed for "twin-twisted and stretched" bars to correspond to a modulus of 22,000,000 p.s.i. The difference between these assumed values of n effects the calculation of stress by only slightly over one per cent.

The deflection of reinforced concrete beams may be a criteria of design in certain cases. Fig. 16 compares the deflections of all the beams at loads of one-third the ultimate strength. The results are somewhat scattered but the average deflection of the beams reinforced with "twin-twisted and stretched" bars ranges from 20 to 35 per cent greater than the average for the beams reinforced with either hard grade or square twisted bars. This increase of deflection agrees well with the fact that the modulus of "twin-twisted and stretched" bars was 25 per cent lower than that of straight bars.

The development of cracks on the tension side of the beam was noted carefully during all the tests. The lower curve in Fig. 17 shows the computed steel stresses for loads at which the first cracks were plainly visible (not hair-line cracks), and the upper curve indicates the computed steel stresses at the appearance of five cracks. For the higher percentages steel the first visible cracks were noted at steel stresses in the neighborhood of 20,000 p.s.i. In this connection the

January 1937 Progress Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete states in Section 875:

In view of the extent to which cracks may develop on the tension face of flexural members the unit tensile stress should be limited to 20,000 lb. p.s.i. in important structural members such as beams, girders, and members of rigid frames.

The number of cracks increased up to the structural yield point, at which load their maximum width was between $\frac{1}{32}$ and $\frac{3}{64}$ in. The type of reinforcing bar had no observable effect upon the number or size of the cracks. Fig. 18 shows the relation between the maximum number of cracks recorded and the ratio of bond area per inch to concrete area.

CONCLUSIONS

1. Both the general "structural yield" and ultimate strength of reinforced concrete beams are proportional to the total yield strength of the tensile reinforcing (yield point stress times steel area) irrespective of the type of bar provided that diagonal tension failure does not occur.

2. No peculiar advantages or disadvantages as tensile reinforcing other than the difference in their respective yield points pertained to any of the types of bars tested except for differences in beam deflection.

3. Beams with "twin-twisted and stretched" bars deflected from 20 to 35 per cent more at a working load of $\frac{1}{3}$ the ultimate than the average of beams with hard grade or square twisted bars.

4. With only three exceptions out of 32 beams tested no cracks were visible to the eye at close range at computed steel stresses under 20,000 p.s.i.

5. Within the range of steel percentages used in the present series of tests (less than 1.00 per cent) the maximum allowable working stresses would be: (a) 40 per cent of the yield-point stress for a factor of safety or 3 with respect to the ultimate strength of the beam; (b) 50 per cent of the yield-point stress for a factor of safety of 2 with respect to the structural yield point.

Discussion, to close in February, 1940 JOURNAL, *should reach A. C. I. Secretary in triplicate by Dec. 1, 1939.*

Closing Discussion

HIGH YIELD POINT STEEL AS TENSION REINFORCEMENT IN BEAMS

by

Bruce Johnston° and Kenneth C. Cox∮

The authors appreciate the attention given to their paper by the discussers. Mr. Withney has used the data in the paper to corroborate his previously published design method. Along the same line the second author, Mr. Cox, has reported similar findings in another paper* as yet unpublished,

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* "The Balanced Design of Rectangular Reinforced Concrete Beams". Thesis prepared for M.S. degree at Lehigh University by Kenneth C. Cox.

which presents other test results as well. Mr. Whitney suggests the definition of 0.004 inches per inch for the yield point because he believes that this total strain is more significant in producing description of the concrete than the yield point determined by the A. S. T.M. offset method. The application of the term "yield point" to the stress at an arbitrary constant deformation would seem questionable. The term "yield point" implies an increased rate of strain with respect to load, which would not necessarily result from the "constant strain limit" of 0.004 inches per inch. In the present series of tests the yield points determined by the A.S.T.M. 0.20 per cent offset method are very close to the strain limit of 0.004 inches suggested by Mr. Whitney. This is a natural result of the fact that most of the yield points are in the neighborhood of 60,000 p.s.i. In the case of "true Isteg" No.1 and No.2, however, the stress at 0.004 in. strain is somewhat less than the reported yield points. This would not improve the agreement in Mr. Whitney's Table 4, although the use of a yield point obtained by the embedded test would, as he suggests, improve his comparison.

Dr. Steinman states on page 1 of his discussion that: "Heat-treates and high-carbon material is uncertain and may be brittle" and "In Isteg steel, the high-yield strength is not obtained by heat treatment nor by increasing the carbon content." The authors wish to point out that no heat-treated steels were used in the investigation. In the hard grade bars the carbon content ranged between 0.35 and 0.46 per cent, as reported by the mill, and reference to Table 2 of the published report shows the hard grade material to be of good uniformity with respect to physical properties as well as possessing good ductility.

Starting in the third paragraph and running through the succeeding five paragraphs Dr. Steinman compares in various ways the behavior of "true" Isteg and that of "improvised" Isteg. Although Dr. Steinman states that the difference in yield points and ultimate strength in these two products may be explained as "largely due to a difference in the grades of steel used prior to twin-twisting and stretching" he believes that "a substantial part of this difference is undoubtedly due to the improvised process of fabrication of the lower value bars".

Adopting Dr. Steinman's terminology of "true Isteg" and "improvised Isteg" it should be emphasized that the "true Isteg" was manufactured from what was originally Intermediate Grade steel and the "improvised Isteg" was manufactured from was was originally structural grade reinforcing steel. Even in comparisons of the same grade of steel variations of ten per cent or more might be expected in the yield point as-rolled. If the twisting and stretching process raised the yield point by a constant percentage the variation would be even greater after twisting. In view of these facts any comparison of the relative merits of "improvised" or "true" twisting processes is incomplete without a statement as to the yield points and ultimate strengths of the straight material prior to twisting.

In the Table on page 4 of Dr. Steinman's discussion it should be pointed out that if Beams I-1-I and I-1-II are omitted from the tabulation, because of their diagonal tension failure, the average ratio of ultimate load to total yield strength will be only 7.7 per cent higher for "true Isteg" than for "improvised Isteg". This may be accounted for by the smaller area of the "true Isteg" bars and their corresponding greater efficienty.

On page 7 Dr. Steinman questions the validity of Fig. 16 in the lower part of the curve because of the limited number of tests reported. The authors agree with Dr. Steinman and did not use this part of the curve in concluding that beams with twin-twisted and stretched bars deflected between 20 and 35 per cent more than beams with straight bars. A careful scaling of Fig. 16 on the 30-kip load line, in the neighborhood of which there are six tests with straight bars and two tests with cold twisted and stretched bars, shows that the deflection is 39 per cent greater at this load for beams with cold-twisted and stretched bars. At a load of 50 kips the deflection is 19.5 per cent greater. The conclusion of the authors in respect to deflection is therefore, a conservative estimate. Dr. Steinman states that increased deflection is seldom a governing consideration in reinforced concrete beam design. The authors believe, however, that visible cracking of concrete may be a serious matter in main beams and girders from an external appearance standpoint. The additional deflection of Isteg reinforced beams coupled with the allowance for higher unit stresses which is made will result in wider visible cracks than for ordinary intermediate reinforcing at normal working stresses.

On page 9 Dr. Steinman cites the high unit stresses reported at the first appearance of cracks and at fine cracks for one beam reinforced with true Isteg. This beam has a percentage of reinforcement of only 0.07 per cent. The authors have based their conclusions with regard to cracking on the greater weight of evidence in the region of 0.8 to 0.9 percentage of reinforcing which is closer to the normal design range.

In the report the authors carefully avoided favoring or discriminating against any particular type of reinforcing steel and listed as their major conclusion the fact that the general "structural yield" and ultimate strength of reinforced concrete beams are proportional to the total yield strength of the tensile reinforcing irrespective of the type of bar and provided that diagonal tension failure does not occur. Dr.

Steinman has raised certain questions in regard to the merit of a particular brand of reinforcing steel. The authors have no objection to any particular type of high yield point steel which has good ductility and uniformity. They believe, however, that very careful consideration should be given to the entire problem of design stresses before a general increase in unit stresses is allowed in any steel used as main reinforcing in beams and girders.