CHAPTER X EFFECTS OF IMPACT ON REINFORCED CONCRETE BEAMS

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SYNOPSIS

The impact resistance of high elastic limit steel bars when used as reinforcement was investigated by means of the Izod tension impact test and by impact tests of reinforced concrete beams

It was found impossible to break the steel in 8- and 16-in beams of 8-ft span reinforced with either one $\frac{1}{2}$-in or one $\frac{5}{8}$-in round bar Repeated blows from the full height available, about 8 ft 4 in, caused increasing deflection and shattering of the concrete, and if continued long enough would rip the bar out of the beam This indicates very clearly the remarkable insurance against collapse provided by even small amounts of reinforcing steel

Another surprising phenomenon was the fact that within the limits of the tests no noticeable difference in impact resistance was displayed by any of the grades of steel used, an almost identical drop being required to break all grades with one blow The beams reinforced with structural steel deflected more than the others, probably thus absorbing the energy that the beams reinforced with stronger steels withstood by direct stress The few shallow beams tested deflected so much that in spite of their lesser mass it was quite as impossible to break the $\frac{1}{2}$ in round bars in them as it was in the deeper specimens

In the Izod tests, no apparent relation between the strength of the Izod tension impact specimen and that of the same steel when used as concrete reinforcement was found.

Introduction One of the questions which has always been raised whenever the use of high yield point steel has been proposed, is that of its resistance to impact. While experimental studies have been made concerning the behavior of isolated specimens of steel under impact, very little has been done in studying its behavior when used as reinforcement.

It has been suggested that the Izod tension impact test should be a reliable guide to the shock resistance of the various grades of steel reinforcement. It has also been suggested that the area under the tension stress-strain curve could well serve a similar purpose. The objectives of the tests conducted during the last two years at the University of Delaware have been. (1) to ascertain the reliability of either of these methods as a criterion for the relative shock resistance of steels of structural, rail, and extra-high elastic limit grades when used as reinforcement, and (2) to determine, by direct test of reinforced concrete beams, the relative merits of these steels when subjected to shock while serving as reinforcement This report, therefore, will be divided into two parts, each covering one phase of the investigation. The author wishes to thank Howard Kent Preston, Jr of the Class of 1937 for his help in reducing much of the data

PART I, IZOD TESTS

From one bar of each grade of steel, a number of standard Izod tension specimens were made First the energy absorbed was determined when a specimen of one grade was broken in one blow, and the elongation was measured and recorded A second specimen of the same grade was then subjected to repeated blows of less but constant energy until broken The elongation after each blow was measured and recorded Then a third specimen of the same grade was subjected to repeated blows of still less but constant energy until broken, elongations after each blow being determined as before, and so on This process was repeated several times to make sure of consistent results, and the whole procedure was repeated for each grade of steel The results are given in Table 1

It will be noted that under repeated 10 ft lb blows, three, four, and six blows steels absorbed vastly more energy than the structural grade steel This is undoubtedly due to the fact that a $2\frac{1}{2}$ ft. lb blow, while causing considerable yield in a structural steel specimen, was barely enough to cause a measurable yield in steels of higher yield point In fact, for the extra-high yield point steel the

TABLE 1

TENSION IMPACT ON IZOD SPECIMENS

	Structural				Rail				Extra-High Yield Point			
Yield Point Y P , Machined[1] Relative	45,000 lb per sq in 45,000 lb per sq in 1 00				57,500 lb per sq in 68,000 lb per sq in 1 51				92,500 lb per sq in 85,000 lb per sq in 1 89			
	Energy Absorbed, ft lb	Elongation, in	Blows	Blows Before Necking	Energy Absorbed, ft lb	Elongation, in	Blows	Blows Before Necking	Energy Absorbed, ft lb	Elongation, in	Blows	Blows Before Necking
When Broken in One Blow	21 0	123	1		27 5	111	1		34 8	133	1	
Relative	1 0				1 31				1 65			
Relative "areas under curve"	1 0				342				455			
Under Blows of												
10 ft lb	25 0	143	3	1	37 0	114	4	Not caught	52 5	141	6	3
Relative	1 0				1 50		-		2 1			
5 ft lb	37 0	190	8	5	58 5	116	12	9	87 8	143	18	13
Relative	1 0				1 58				2 37			
2 5 ft lb	56 5	233	23	18	141 5	169	57	55	698 0	141	279	175
Relative	1 0				2 5				12 36			
Tension Test in Usual Machine (Slow)		190				110				130		
Smallest Blow to Cause Yield					1 50				1 75			

[1] Machined to same shape as Izod specimen

were required to cause rupture in the structural, rail, and extra-high yield point specimens respectively. Under 5 ft lb blows the number required to cause fracture was relatively nearly the same, although slightly greater the higher the yield point Under repeated $2\frac{1}{2}$ ft lb blows, however, the higher yield point condition was fast approaching one of fatigue testing, most of the elongation occurring during the first 25 blows Later, until near the end, the elongation under increments of 50 more blows was scarcely measurable Undoubtedly, if other specimens of structural and rail steel had been tested under repetitive loads of less than 2½ ft. lb each, more commensurate with their yield points, both the number of blows and the total energy absorbed by each of these grades also would have been greatly increased

One rather interesting feature was that with the extra-high yield point steel, the total elongation, whether under impact or under the slow motion of the usual tension-testing machine, varied but little, whereas structural steel, and to a lesser degree rail steel, was deformed much more under repeated blows of low length. This is undoubtedly due to the fact that in the Izod specimen, the necked section occupied a relatively greater proportion of the gauge length. The interesting feature is, however, that for a given elongation, a greater unit stress was sustained by the Izod specimen Unless it be due to cold working during the process of machining the Izod specimen, the author can offer no explanation of this phenomenon

The same behavior is noticeable with rail steel, the Izod specimens having a

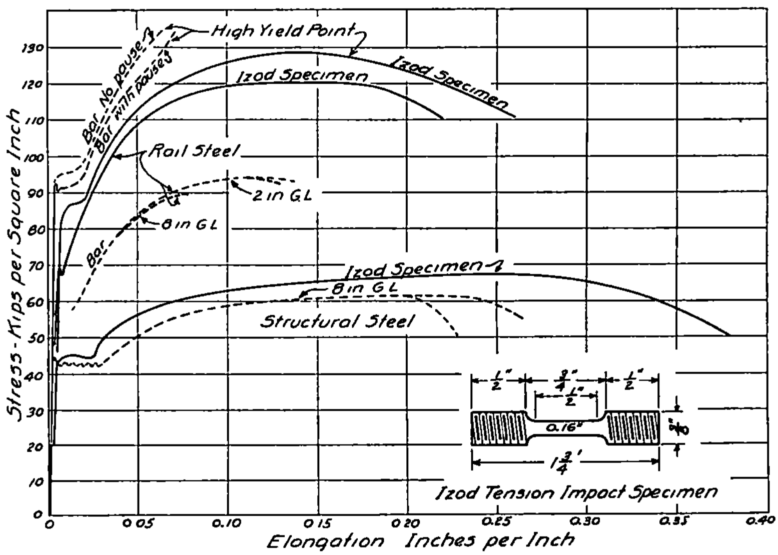

Figure 1

energy than in the usual tension test The significance of this feature will become apparent in the discussion of the beam tests

In order to ascertain whether the steel in the standard Izod specimen was representative of the steel in the bars, companion stress-strain tests were made between an Izod specimen and unmachined coupons cut from the same bar. These data are plotted on Figure 1 For example, it will be noted that in the case of structural steel, the Izod specimen elongated fully 50 per cent more than the unmachined steel in an 8 in. gauge greater unit elongation at failure and sustaining a higher tensile unit strength for a given elongation With the extra-high yield point steel, however, a curious reversal in performance was noted While the unit elongation was much greater at failure with the Izod specimen, the unit stress for a given unit elongation was always less. Perhaps it should be remarked that in conducting these tension tests, every load increment was allowed to remain until all yield had ceased, or for at least 15 minutes As a check, a few specimens of extra high yield point steel were tested in the usual manner without stopping the testing machine The curve so determined is labelled "Bar—No pause" When we hear, therefore, that the total energy absorbed by an impact specimen is equal to the "area under the curve," the question may well be asked, "Under what curve?" Cold working and dissipation of heat make comparisons particularly difficult to draw

PART II, BEAM TESTS

Elastic Analysis The study of the elastic resilience of beams is not new Hodgkinson, in 1833, made a rather elaborate experimental investigation of the resistance of flexural members to transverse impact, while in 1849, Homersham Cox published the theoretical study which still forms the basis of such discussions [1]

We have, for example, the following expression for the deflection of the beam when the mass of the beam itself is disregarded

$$\Delta = \Delta_1 + \Delta_1 \sqrt{1 + \frac{2h}{\Delta_1}} \qquad (1)$$

in which Δ is the maximum deflection of the beam due to impact, Δ_1 is the deflection caused by the weight, W, applied as a static load. From this, the equation for stress in a beam due to an energy load when the mass of the beam is disregarded, is[2]

$$S = S_1 + S_1 \sqrt{1 + \frac{2h}{\Delta_1}} \qquad (2)$$

When the mass of the beam is not disregarded, equation (3) is an expression for its deflection

$$\Delta = \Delta_1 + \Delta_1 \sqrt{1 + \frac{2h}{\Delta_1} \frac{1}{1 + \frac{17W_1}{35W}}} \qquad (3)$$

in which W_1 is the weight of the beam and W the weight of the falling hammer.[3] Stresses due to this deflection may be found from equation (4)

$$S = S_1 + S_1 \sqrt{1 + \frac{2h}{\Delta_1} \frac{1}{1 + \frac{17W_1}{35W}}} \qquad (4)$$

For a beam of constant cross section and with a given falling weight, S_1, Δ_1, and $\frac{1}{1 + \frac{17W_1}{35W}}$ are constants If the latter be represented by m, we have, by transposition

$$S - S_1 = S_1 \sqrt{1 + \frac{2h}{\Delta_1} m} \qquad (5)$$

Squaring

$$(S - S_1)^2 = S_1^2 \left(1 + \frac{2h}{\Delta_1} m\right) = \frac{2mS_1^2}{\Delta_1}\left(h + \frac{\Delta_1}{2m}\right) \qquad (6)$$

which is in the standard form of the parabola

$$(S - S_1)^2 = c(y - k) \qquad (7)$$

in which

$$c = \frac{2mS_1^2}{\Delta_1}$$

and

$$k = \frac{-\Delta_1}{2m}$$

The graph of equation (6) is shown in Figure 2, which gives the relation between the height of fall and the corresponding unit stress in the extreme fiber of the flexed member

Since the usual equations for stresses in reinforced concrete members are based on the same assumptions as those in members of a homogeneous material, the

[1] Todhunter and Pearson, p 771

[2] Seely, Resistance of Materials, 2nd Ed, p 286

[3] After Timoshenko, Strength of Materials, part I, p 313

same equation may be used in computing the steel stress, and Figure 2 also shows the relation between unit stress in the reinforcement and height of fall This curve remains below the horizontal axis from $S = O$ to $S = 2S_1$, because the stress caused by a "suddenly applied load," which may be described as an impact load in which $h = O$, is $2S_1$

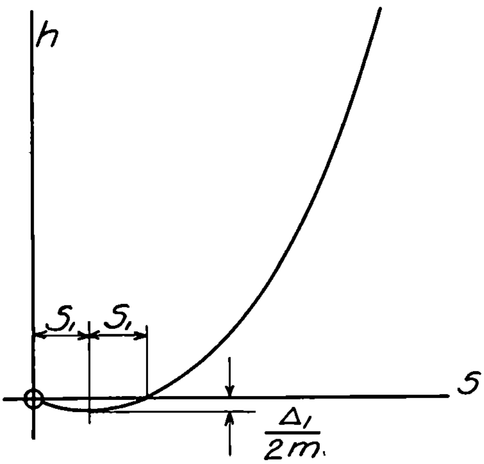

Figure 2

Now in a reinforced concrete simple beam carrying a single concentrated load at midspan,

$$A_s S j d = \frac{PL}{4}$$

or

$$P = \frac{4A_s j d}{L} S$$

which may be represented as

$$P = cS \tag{8}$$

Combining equations (6) and (8) we have

$$\left(\frac{P}{c} - S_1\right)^2 = \frac{2mS_1^2}{\Delta_1}\left(h + \frac{\Delta_1}{2m}\right)$$

or

$$\frac{1}{c^2}(P - cS_1)^2 = \frac{2mS_1^2}{\Delta_1}\left(h + \frac{\Delta_1}{2m}\right)$$

from which

$$(P - cS_1)^2 = \frac{2mS_1^2c^2}{\Delta_1}\left(h + \frac{\Delta_1}{2m}\right)$$

but

$$cS_1 = \left(\frac{4A_s j d}{L}\right)\left(\frac{WL}{4A_s j d}\right) = W$$

Substituting in the previous equation, we have

$$\begin{aligned}(P - W)^2 &= \frac{2mW^2}{\Delta_1}\left(h + \frac{\Delta_1}{2m}\right)\\ &= W^2\left(\frac{2mh}{\Delta_1} + 1\right)\end{aligned} \tag{9}$$

which is in the standard form of a parabola Equation (9) gives the relation between the static load P necessary to produce a given stress and the height h from which a given weight W must fall to produce the same stress in a given beam Figure 3 is a graphical representation of equation (9)

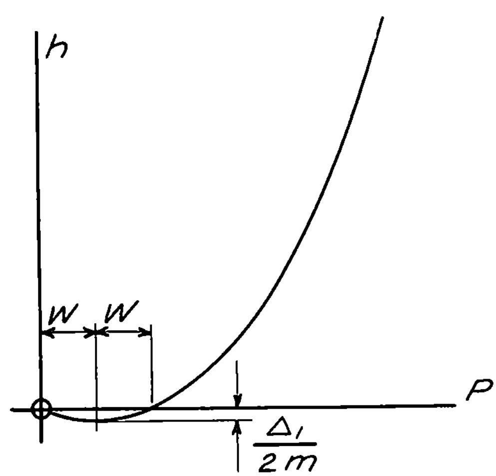

Figure 3

In predicting deflections from equation (3) or stresses from equation (4), it remains only to find a value for Δ_1.

This may be done quite satisfactorily by means of Maney's equation[4]

$$\Delta_1 = c \frac{L^2}{d} (e_c + e_s) \qquad (10)$$

in which

e_c = unit deformation in extreme fiber of concrete $= \frac{f_c}{E_c}$

e_s = unit deformation in steel $= \frac{f_s}{E_s}$

c = a constant depending upon the type of loading, in this case equal to 0 0833

Conversely, Maney's equation would be useful in finding fiber stresses when the measured deflection under static load is known

The foregoing study is, of course, applicable only when the elastic limits of the component materials are not exceeded It is included primarily to show how far the reserve of ultimate strength under the influence of impact exceeds what might be expected from an elastic analysis This is dealt with further in the discussion of results.

Concrete Mixes. Lehigh portland cement was used in all tests. The coarse aggregate was nominal ¾-in. Delaware River gravel having a fineness modulus of 5 41, and the fine aggregate was Delaware River sand with a fineness modulus of 2 93 Both sand and gravel were composed of clean, hard grains with a very small percentage of dirt

In a few preliminary beams concrete with a cylinder strength of 6000 lb per sq in was tried, but the resulting beams were no stronger than those of 3000 lb concrete For the remainder of the program 3000 lb concrete was used

Reinforcement The majority of the beams were reinforced with round bars of either structural or rail steel. A few were reinforced with a silico-manganese billet steel having an extra-high yield point This steel contained C = 0 54 per cent, Si = 1 96 per cent, and Mn = 86 per cent, thus falling under classification SAE 9250 A few others were reinforced with another hard grade billet steel having a yield point of 65,400 lb per sq in Owing to the small size of the bar yield points were somewhat higher than usually specified. The average values are shown in Table 2 The ¼ in bars of structural grade were smooth, all others being deformed

Test Beams Nearly all the beams were 10 in wide and 16 in deep. A few were 10 in wide and 8 in deep In all deep beams the reinforcement was supported on 1-in blocks so as to give 1 in. full coverage In the shallow beams the reinforcement had ½ in. coverage. All beams were 9 ft. long and were tested on an 8 ft. span

In the top of each beam was embedded a ¾ in steel plate 6 in square to prevent local shattering under the blows

At first the beams were handled by means of lifting hooks—pieces of ¼ in round bars shaped like hairpins—embedded about 6 in into the upper surface of the beams at the outer quarter points of the span Since cracks always appeared at these points under relatively light blows the use of such hooks was abandoned until it was demonstrated that the cracking was due to other causes

Testing Apparatus. Nearly all impact tests on flexural members have been conducted in machines consisting essentially of a large pendulum, the bob of which strikes the beam horizontally at mid-span, the beam lying on its side Since an impact momentum of at least 4,000 ft lb was desired, the construction of such an apparatus would have been exceedingly costly Therefore a drop

[4] See paper by G A Maney, 17th annual meeting of the A S T M

weight type somewhat similar to that used in testing rails was used. The use of this apparatus is attended with two serious difficulties First, the weight remains in contact with the specimen, thus damping its free oscillation and, second, in order to determine the breaking load with any degree of exactness, many specimens had to be broken with various heights of fall The results obtained, however, are within the limits of accuracy that might be expected in testing non-homogeneous members of the type under investigation. In the testing apparatus a 560 lb ram slides freely in guides fastened to an A frame which is in turn supported on a concrete base 3 ft wide, 10 ft. long, and 5 ft deep. The base weighs approximately 22,500 lbs — about 40 times the weight of the ram. The beams were supported on the ends of the webs of two 3-ft sections of 14-in. WF 108 lb steel beams, 8 ft center to center, embedded vertically half their length in the concrete base To prevent the webs from cutting into the concrete, each test beam was bedded in plaster of paris at each end on a $\frac{3}{4}$ in by 2 in. steel plate.

The ram was raised by means of a chain hoist and released by a trip hook operated by a cord The lower surface of the ram was planed flat and impinged on a 6 by 6 by 2-in steel block rounded on the upper surface to ensure the central application of the blow. This auxiliary block was placed on top of the steel plate previously embedded in the beam. Figure 4 is a general view of the testing frame with a beam in place

Deflection Measurements The study of the Izod specimens having revealed the fact that the minimum total energy was absorbed when the specimen was broken by a single blow, it was deemed advisable to attempt to break the test beams also by means of a single blow The results of such a procedure, too, would be of more immediate value to the man engaged in construction, for he is worried primarily about the possibility of the breaking of brittle reinforcement

TABLE 2

BEAMS 10 IN BY 16 IN ON 8-FT SPAN

1	2	3	4	5	6	7	8	9	10	11	12	13
Reinf Round Bars	%	Steel Stress Due to			Blow Required to Rupture Steel	Blow Required for f_s = 45000[4]	Yield Points			Static Concentrated Load Required to Produce Failure, lbs [5]		
		Wt of Beam	Ram Static	Total			Struct	Rail	Extra High	Struct	Rail	Extra High
1-$\frac{1}{4}$ in	03	24700	20100	45100	9 in [1]	—	45000	86000	112000	680	1930	2720
1-$\frac{3}{8}$ in	07	10900	9000	19900	48 in [2]	1 10″	45000	61000	92500	2690	3870	6200
3-$\frac{1}{4}$ in	09	8235	6767	15000	54 in [3]	2 06″	45000	86000	112000	3420	5490	9550
1-$\frac{1}{2}$ in	13	6175	5075	11250	—	4 58″	45000	57500	92500	4780	6300	10560
1-$\frac{5}{8}$ in		3950	3225	7175	—	16 10″	45000	55400	—	7860	9820	

[1] 12-in blow for the smooth structural steel bars Smooth bars permitted greater deflection
[2] 46-in blow for structural steel
[3] Rail steel Structural steel a little less
[4] Computed by means of Equation (9)
[5] Computed, see "Results "

For this reason it was not possible to take many refined deflection readings It was impossible in those beams which broke on the application of a single blow, and for the others measurement by means of a foot rule was accurate enough. To make the record more complete, however, one beam was reinforced with one $\frac{5}{8}$-in round deformed rail steel bar. A stylus attached to its side traced on a smoked glass plate the deflections caused by

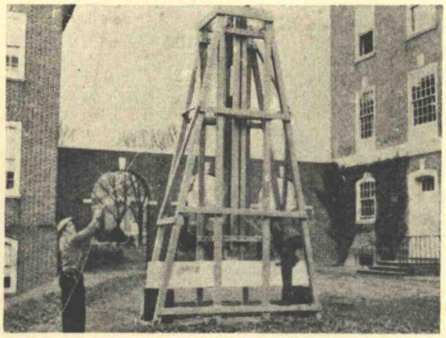

Figure 4. Testing Frame with Beam in Place

blows of increasing magnitude. The result is shown in Figure 5. The deflection under the static weight of the ram is barely discernible, while the effect of 2-in., 12-in., and 36-in. blows is prominent. The recovery when the weight was lifted after the 36-in. blow may also be seen. It should be noted that even after the 2-in. blow a measurable permanent deflection was noticeable.

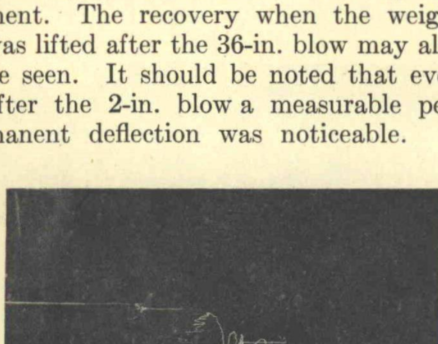

Figure 5. Deflections under Blows of Increasing Magnitude

Such measurements are hard to make for conditions beyond the elastic range of stress. The extreme suddenness of an impact load causes the stylus to vibrate laterally, and permanent elongation of the bottom of the beam (as much as 2 in. in some of our beams) sometimes results in longitudinal displacement.

Method of Testing. A blow with a fall of $1\frac{1}{2}$ in. would generally cause the formation of the first vertical crack at midspan, but in order to avoid the necessity of repeating this blow each beam was struck a preliminary 2-in. blow. As may be seen from Table 2, a greater blow would have seriously stressed the steel in beams with little reinforcement. It is important, in studying the results, to realize how small an energy load was sufficient to rupture the concrete.

After the preliminary blow we attempted to strike the beam a blow that would just break it. On a similar beam we tried to deliver a blow that would almost break it, in this manner closely bracketing the breaking load. If the beam was not broken, it was subjected to repeated blows till rupture occurred, if within the capacity of the apparatus.

Motion pictures were taken at four times normal speed so as to slow down the action when projected for study. Even then the extreme suddenness of the phenomenon of impact is hard to realize.

It was found impossible to break the steel in the beams reinforced with either one $\frac{1}{2}$-in. or one $\frac{5}{8}$-in. round bar. Repeated blows from the full height available, about 8 ft. 4 in. caused increasing deflection and shattering of the concrete, and if continued long enough would rip the bar out of the beam as shown in Figure 6. This indicates very clearly the remarkable insurance against collapse provided by even small amounts of reinforcing steel.

Another surprising phenomenon was the fact that within the limits of the tests no noticeable difference in impact resistance was displayed by any of the grades of steel used, an almost identical drop being required to break all grades.

The beams reinforced with structural steel deflected more than the others probably thus absorbing the energy that the beams reinforced with stronger steels withstood by direct stress. The few shallow beams tested deflected so much that in spite of their less mass it was quite as impossible to break the ½-in. round bars in them as it was in the deeper specimens. Figures 7 and 8 show a beam reinforced with one ½-in. round structural grade bar after the first and second full height blows. Figures 9 and 10 are similar views of a beam reinforced with one ½-in. round rail steel bar.

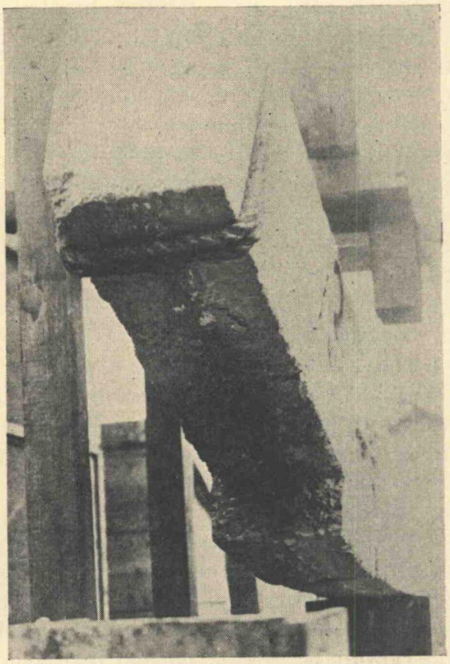

Figure 6. Bars Ripped from Beam under Repeated Blows from Full Height Available

Results

In columns 3, 4 and 5 of Table 2 are given the computed steel stresses due to the dead weight of the cracked beam with the ram resting on it as a static load. The yield points of the various reinforcing steels are given in columns 8, 9 and 10. In columns 11, 12 and 13 are given the static, concentrated, centrally applied loads which would be required, in addition to the weight of the beam, to produce failure, on the assumption that slow failure would follow when the computed stress was 10 per cent higher than the yield points tabulated.

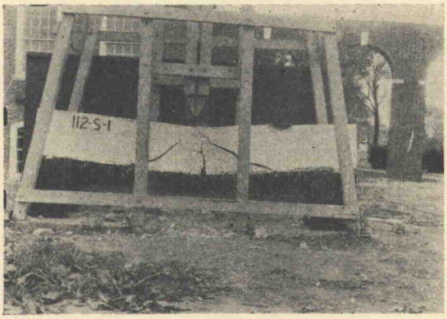

Figure 7. After First Full Height Blow. Beam reinforced with one ½-in. round structural grade bar.

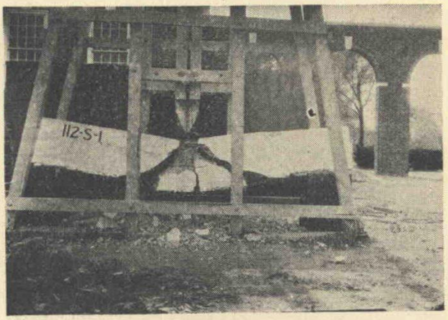

Figure 8. After Second Full Height Blow. Beam reinforced with one ½-in. round structural grade bar.

In column 7 are given the heights of drop required to produce a stress of 45000 lb. per sq. in. in the reinforcement, as computed by equation (9). For the beams reinforced with one ¼-in. round bar the value is negative, which is confirmed by comparing the unit stresses in Cols. 5 and 7. The height of drop to cause actual rupture of the steel compared to the drop required to stress the steel to the yield point of structural steel may be visualized by comparing columns 7 and 6. Here again we see the enormous reserve resistance to impact in the reinforcement,

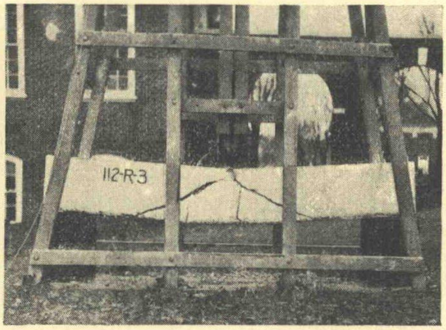

Figure 9. After First Full Height Blow. Beam reinforced with one ½-in. round rail steal bar.

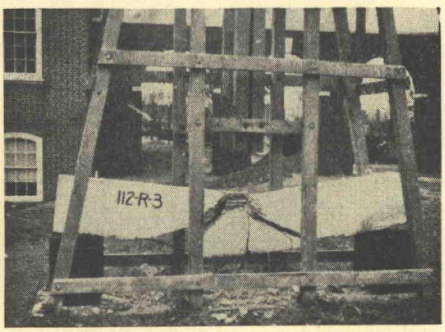

Figure 10. After Second Full Height Blow. Beam reinforced with one ½-in. round rail steel bar.

when stressed beyond the yield point. The ¼-in. bars of structural steel, for example, were stressed to their yield point while carrying the ram as a static load, yet withstood a 12-in. blow.

This may be visualized in still another way. Suppose that one of the beams had been reinforced with four ⅝-in. round bars (¾ of 1 per cent). The deflection under the static weight of the ram, by equation (10), equals 0.0029 in. Substituting this figure for Δ_1 in equation (3), and assuming a 6-ft. drop, the deflection under the impact load would be 0.41 in. The corresponding unit stresses, computed by the relation $\frac{\Delta}{\Delta_1} = \frac{S}{S_1}$, are 150,000 lb. per sq. in. in the steel and 6020 in the concrete. Yet it was impossible to break one ½-in. round bar.

In every beam, under even a moderately heavy blow, a vertical crack formed at each outer quarter point, extending from top to bottom. It was at first thought that these cracks might be due to the presence of the lifting hooks, but the same cracks formed in beams without hooks. It may be possible that a wave action generated by the sudden load at midspan was responsible. Without more information it would be useless to speculate whether this wave motion proceeded from midspan toward the ends or consisted of standing waves with the nodes at the ⅛ points of the span.

As far as the Izod tests are concerned, there is no apparent relation between the strength of the Izod tension impact specimen and that of the same steel when used as concrete reinforcement.

With the nature of the problem thus disclosed, we hope to be able in the near future to repeat the foregoing tests using a much heavier ram. In the meantime it is hoped that engineers who are concerned with the impact resistance of rail and other high elastic limit steels may find comfort in the results of these tests.