UNIVERSITY OF MANITOBA

DEPARTMENT OF CIVIL ENGINEERING

MASTER OF SCIENCE THESIS

April 1969.

THE EFFECTS OF COLD WORKING ON THE MACROSCOPIC AND MICROSCOPIC PROPERTIES OF REINFORCING STEELS

Submitted as a partial fulfillment of requirements for the degree: Master of Science.

Submitted by: Tibor Pataky

Tibor Pataky

ACKNOWLEDGEMENTS:

The author would like to extend his grateful thanks to all who have contributed, in any way, in the preparation of this thesis.

Thanks are due in particular to the author's advisor, Professor R. Lazar for his constant attention and assistance.

Thanks are also extended to Dr. K. Tangri for his guidance in metallography; to Messrs. A. Sopotiuk & E. Lemke for their invaluable help in the laboratories. To Mr. D. Lunder for the casting of concrete blocks.

TABLE OF CONTENTS

PURPOSE:

The purpose of the studies conducted at the University of Manitoba in connection with cold bending (cold working) of reinforcing steel was to determine load carrying capacity and mode of failure when such bent bars are subjected to "shear loading" or "direct tensile loading".

INTRODUCTION:

It is difficult to imagine reinforced concrete or pre-stressed concrete structures that would contain straight reinforcing only.

The stirrups in the beams, the lateral ties in the columns, the hooks provided for anchorage, bent bars in webs of beams, and bent bars welded to masonry plates for anchorage of pre-cast concrete connections, are only a few examples of bent bar applications.

One would not hesitate for a moment to specify a hook at the end of a straight bar if embedment requirements set forth by one code or another cannot be met due to physical limitations.

Due to rapid development of the materials sciences there are quite a variety of reinforcing steels available.

The range of minimum yield strengths cover a fairly wide field from 30,000 p.s.i. to 75,000 p.s.i.

The two most popular grades of steel are probably the A.ST.M. A15 intermediate grade with 40,000p.s.i. minimum yield and the A.S.T.M. A432 hard grade steel with 60,000 p.s.i. minimum yield strength.

The later of the two steels mentioned is gaining wide

popularity due to its higher allowable stress in tension and especially in compression.
It is needless to say that the use of high strength reinforcing results in saving of reinforcing, increased space due to reduced column sections, and reduction of story height in multi-story buildings.
The various concrete, prestressed concrete and reinforcing steel institute codes provide the designer with fairly safe and moderately conservative guides and tools with which he can design safe and economical structures.
Thanks to the general acceptance of the various codes by the designers of concrete structures failures of violent nature are a rare occurrance.
Notwithstanding the relatively blemishless record of reinforcing in service the author is aware of a few situations where failure of reinforcing at the bend manifested itself in a very sudden and brittle type of failure.
The very fact that designers are using the higher strength reinforcing prompts the author to ask these questions:

(i) Are the bends in reinforcing steel capable of doing the job they were intended for in spite of the fact that a fair percentage of them actually fail during bending?

(ii) What are capacities of precast concrete column-to-beam connections with bent bars welded to plates that provide seating for beams?

(iii) Or, what are the actual capacities of hooks and bends once in the confines of concrete?

The intent of this paper is to answer the foregoing questions and hopefully, eliminate some of the hazardeous practices that may still exist in the design of precast concrete connections.

In addition, the paper will review some of the limitations set on "hooks" by the A.C.I. Code of 1963.

SCOPE OF STUDY:

The study encompassed two sets of mechanical testing:

(1) Bent bars welded to plates

(2) Direct tensile testing of "hooks"

and one set of micro-analysis of bends.

(1) Bent Bars Welded to Plates

This series of tests involved bending two grades of reinforcing, A.S.T.M. A15, and A.S.T.M. A432, into "U" shapes. The straight portion of the "U" was welded onto one edge of a plate as shown in figure 1. Then the free standing legs of the "U" were embedded in concrete having the welded portion of the bar flush with the top of the concrete. The testing apparatus shown on page 41 was used to pull the anchorages out of the concrete while ultimate load and mode of fracture were recorded.

Stress and strain analysis were done for a number of bars in an attempt to varify the actual behaviour of the bent-welded bars under, what one might call, "shear loading".

(2) 90 degree hooks embedded in concrete.

Again, two grades of reinforcing, A432 and A15, were used.

Reinforcing bars were bent 90 degrees about pin

diameters specified by A.C.I. 318 - 63 801(b).
The hooks were embedded in blocks of concrete.
To be able to determine the capacity of the hook only, the straight portion of the bars were wrapped in paper to prevent bonding to the concrete.
See details of specimen in figure 7, on page 47.
The bars were then subjected to "direct tensile loading" until failure occured.
Ultimate loads and modes of failure were recorded.

Micro - Analysis of Bends

This examination involved taking longitudinal sections out of bends of various sizes of reinforcing bars from both types of steels.
After all specimen were polished and etched various techniques were used to detect possible "microcracks" in the extreme tensile fibers.
Compression and tension zones were compared.
A number of photographs were also taken showing the effect of cold working on both the tensile and compressive zones. See micrographs 1 to 4.

DESCRIPTION OF TESTS AND DISCUSSION OF RESULTS

(1) Bent Bars Welded to Plates

To simulate actual conditions that occur in precast concrete connections bars of different grades of steel in different sizes were bent, as shown in figure 1, observing the minimum radii of bend allowed by the A.C.I. code. Low hydrogen electrodes were used in order to reduce hydrogen embrittlement of the weld due to entrapping hydrogen gasses within the weld metal. The welding was done at room temp. by qualified welders as specified by A.W.S. and the Canadian Welding Bureau.

In precast concrete connections bent reinforcing when welded to masonry plates is usually intended to carry only loading perpendicular to the plane of the bent-welded bars. It is easy to see, however, that say a beam seat on a precast column is subjected not only to vertical beam reaction but also to horizontal frictional loading which may be either tensile or compressive depending on whether contraction or expansion is taking place in the beam.

The horizontal load on beam seats could easily be as high as 50% of the vertical load depending on the roughness of the sliding surfaces.

In this series of tests the effect of the vertical reaction on the bent-welded bars was excluded and only the horizontal tensile load was simulated to occur.

The thickness of the plate was selected to be such that it was always at least $\frac{1}{4}$ of an inch to $\frac{1}{2}$ of an inch thicker than the diameter of the bars to facilitate deposition of weld metal.

The results of the test for the bent - welded condition are shown in Table 1, on page 42.

There did not appear to be any deformation prior to fracture other than a slight displacement of the welded portion of the bar accompanied by spalling of the concrete.

Shortly after spalling of concrete, very sudden failure occured invariably for every assembly.

Only a few double fractures and a few partial double fractures (where one leg failed completely and the other failed partially) have occured.

Approximately 20% of the fractures appeared clearly a distance away from the ends of the welds having the majority break at the ends of the welds.

For location of fracture planes see figures 4 & 5, on pages 44, and 45.

The appearance of the fracture surfaces clearly identified the mode of failure to be of the cleavage (I)* type, appearing granular, thus pointing to a brittle type of failure. See photographs 1 to 4.

The load carrying capacity of the bent-welded bars showed a marked reduction compared to the straight bar capacity, as can be seen in Table 1 & Table 2, on page 42.

Looking first at the ultimate capacity of bent-welded bars in the A.S.T.M. A432 steel one can notice a strength reduction of 70 - 80% as compared to straight bar capacities. The milder steel, in A.S.T.M. A15, showed that the bent-welded bar capacity reduction was only 47 - 69% of the straight bar capacity, see figure 3, on page 43.

The explanation for the brittle behavior and the large reduction in load carrying capacity lies in the following:

(i) cold working of material

(ii) residual stresses due to welding

(iii) residual stresses due to inelastic bending

(iv) triaxial state of stress when load is applied.

---

*Roman numerals in perenthesis stand for reference identification. Please see list of references on page 39.

(i) Let us look at the first and most dominant reason for reduced capacity and brittleness.
Cold working or plastic working of steel can be examined at the "macroscopic" and "microscopic" levels.
Only the "macroscopic" aspect of plastic working shall be examined now, while the microscopic aspects will be discussed under the appropriate heading later in this paper.
Macroscopic plasticity begins with certain observations concerning plastic deformation of polycrystalline metals in simple mechanical tests, such as the tension test, and from the results of the direct tension test proceeds to develop some theory of gross plastic flow.
In the macroscopic viewpoint, the metal is thought of as a "continuum" (II) having properties such as density, stress and velocity at all points within its outer surface.
Stress and strain play a central role in continuum theory and describe in an average way the forces between the atoms in the crystal lattice and the deformation of the lattice respectively.
Stress and strain are described by "tensors".
One of the properties of tensors is that they have three principal values corresponding to three perpendicular planes through a point. These planes

of course, do not contain shear stresses.

The theory of macroscopic plasticity depends very largely on three macroscopic observations concerning stress and strain rate.

Firstly, that volume remains essentially constant during gross plastic deformation.

Secondly, that yielding occurs at any point only after the maximum shear stress in some direction on some plane attains a critical value.

Thirdly, that the direction of greatest shear strain rate coincide with the directions of greatest shear stress. It can also be noted that plastic flow of the metal under stress must be possible if work-hardening is to result, (III) (IV).

Under the extreme case, if a metal were to be subjected to equal triaxial tensile forces, there could be no flow and hence any such metal should behave as if it were completely brittle.

Also, a material which is severly work-hardened may behave like a brittle material, (III) (V).

It is believed by the author that reinforcing bars when bent to the minimum radii specified by, say the A.C.I. Code, undergo very severe work-hardening process.

To illustrate the point, let us take a one inch diameter bar, bend it about a six inch diameter pin, then the tensile and compressive strains at the extreme fibers would be in the order of 14.3%
The strain at yielding for the mild steel is in the order of 0.16% and for the hard grade steel it is approximately 0.20% . This implies that only approximately 1.5% of the steel area has not gone under plastic deformation for the mild steel and only about 2.0% of the steel remains elastic for the hard grade steel.

(ii) It is a known fact that welding introduces residual stresses. Welding stresses are not caused soleley by the freezing and shrinkage of the weld metal as shown in figure 6, on page 46.
As a matter of fact, stresses of the same nature and magnitude are introduced in the edge of the plate and the weld metal in a longitudinal direction of the bar. The reason for this is as follows:
When the arc is moved along the edge of the plate, the metal in the heated region expands. Since only a small volume is hot at any time, the expansion of the heated material is restrained along the edge (I).

Thus expansion or plastic flow can occur only in the two unrestrained directions normal to the edge, and so upsetting occurs whenever temperatures are high enough to induce compressive yielding. The heating, being done progressively along the plate edge from one end to the other, and the material along this entire length becomes thicker.

After cooling, the upset edge is too short to conform to the adjacent portion of the plate, thus residual tensile stresses are induced along the edge of the plate.

To sum up residual stresses caused by welding one can say that there are longitudinal and transverse stress fields set up in the weld metal and in the fibers of the bent-welded bar adjacent to the weld.

(iii) When a bar is bent beyond the elastic limit, permanent set is produced, and the deformation does not vanish after the removal of the load.

The fibers which have suffered a permanent set prevent the elastically stressed fibers from recovering their initial length after unloading, and in this way residual stresses are produced (VI).

It is assumed that the material, which is stressed beyond the yield point, follows Hook's law during unloading resulting in bending stresses that follow the linear law as indicated in figure 6(a), on page 46.

When the rectangular loading and the triangular unloading stress diagrams are superimposed on one another the areas in between the two stress curves represent the residual stresses within the bar.

Since the deformations of the reinforcing bars due to cold bending is so great, that only 1.5 to 2.0% of the cross-sectional area remains elastic, and that a large percentage of the cross-sectional area of the bars have undergone strain hardening, rebound is greatly inhibited by dislocations within the crystal structure of the steel.

Residual stresses caused by inelastic bending in the opinion of the author is not as critical as it may appear from figure 6(a), which was prepared after S. Timoshenko's reasoning, which may have been based upon bars or beams bent passed the yield point but not as far that work hardening could have taken place.

However small these stresses may be their existance can easily be verified by observing the actual process of cold bending. It is a known fact that for instance if a 90 degree bend is required the operator of the bending machine bends the bars a few degrees beyond 90, knowing from his experience that the bars will rebound or "spring back" a certain amount as soon as the bending forces are removed.

(iv) Finally, when the bent-welded bars were subjected to a loading condition that simulated horizontal load on column-to-beam brackets, the bent-welded bars were already subjected to cold working, residual stresses due to welding and inelastic bending.

It was not surprising at all that only a relatively small amount of additional strain energy could be absorbed by the bent-welded bars.

The average applied shear stress at the end of the weld for the different bars and grades of steel varied between 25,000 p.si. and 37,000 p.si. at the ultimate load. It is also a know fact that a material like steel if loaded by shear alone to a stress level of approximately 57.7% of the yield point in tension then the material failes by diagonal tension. Most of the bent-welded bars failed at a shear stress much less than 57% of the yield strength (which yield strength varied from 56,000 p.si. average for the mild steel to 71,800 p.s.i. for the hard grade steel), indicating that strain hardening, residual stresses and other effects due to welding are a contributing factor in the marked reduction of strength.

The granular appearance as can be seen in photographs 1 to 4, may be explained by the fact that in the outside of the bend where tension governs during bending the grains are elongated and drawn tightly against one another. When a load is applied in a direction perpendicular to the elongated body of the grains there is just no possibility for deformation and the microcracks created by plastic flow quickly develop into a very unstable cleavage type of crack.

Thus the reason for the sudden failure is threefold: restraint (residual stresses), possible microcracks, and high applied shear stresses.

Microcracks will be discussed at the end of this paper.

(2) 90 degree Hooks Embedded in Concrete

According to A.C.I. 318 - 63 801(a) a "standard hook" may be defined as a "90 degree turn plus an extension of at least 12 bar diameters at the free end of the bar".

In this series of test two grades of reinforcing steel A432 and A15 were bent 90 degrees to minimum radii specified for #4, #5 and #6 bars.

An extension of only four bar diameters were provided at the free ends of the bars as specified by the British Standards.

The straight portion of the bars were wrapped in paper to prevent bond developing between concrete and reinforcing. The hooks were embedded as shown in figure 7, on page 47.

The hooks were attemted to be pulled out when concrete strength of the blocks reached 3,500 p.s.i. minimum.

Failure was anticipated at approximately 50% of the straight bar capacity and it was thought to occur somewhere in the bend.

A surprising thing happened. All the eighteen hooks tested, developed the full strengths of the bars and failure occured in a normal cup-and-cone fashion in the straight portion of the bars on the outside of the blocks.

Actually, two out of the eighteen blocks split during the test, but at a fairly high load of 44,250 lbs. and 37,500 lbs.

Both these blocks had an A.S.T.M. A432 hard grade #6 bar embedded in them and the tensile stresses in the straight portion of the hook were 100,000 p.s.i. and 85,000 p.si. respectively at the time when the blocks split.

Therefore, for all practical purposes one can assume that all the hooks developed the straight bar capacities in spite of the two blocks splitting.

To be absolutely sure that there was not any bond developed along the straight portion of the bars a few blocks were split open after the test to examine the wrapping, the concrete and the bar.

The wrapping was found to be loose, soggy and easily removable closing out any chance for any appriciable load transfer. (Please see photograph 11, page 53.)

According to the "Commentary" to the A.C.I. 318-63:

"No research to establish minimum bend radii related to stress in bar, or concrete strength to prevent crushing of concrete within the bend was available for the present bars".

in the light of the statement in the "Commentary" and the test results discussed earlier there appears to be some need for research to rationalize the relationship between

(i) radii of bends and capacity of hooks

(ii) radii of bends and behavior of concrete at the inside of the bend

(iii) capacity of hooks and extension length.

If the eighteen hooks tested by the author are only an indication of what might be expected of a comprehensive series of test, then the question may be asked: Why is it necessary to bend bars 180° when 90° would do?

Why is the 12 bar diameters extension required for a 90 degree bend when probably four bar diameters extension is sufficient?

There is an interesting observation that can be made in connection with the allocation of load carrying capacity to the various parts of the hook. If the results of the "bent-welded" bars test are examined closely, one can see that for the #4, #5 and #6 bars in question the ultimate shear load varied between $1/4$ and $1/3$ of the straight bar capacity. Having this in mind it is easy to see that the extensions of the hooks in the test could not have carried more than $1/4$ to $1/3$ since this would have resulted in a fracture.

Thus, the conclusion is that at least $2/3$ to $3/4$ of the load is developed in the 90 degree bend itself! Of course in order to be able to develop large loads in hooks one must provide sufficient concrete in volume as well as in strength and/or lateral reinforcing to prevent bursting or splitting of concrete.

In summary one may conclude that for the eighteen hooks tested, under the circumstances as described herein, all the hooks developed the full strengths of the bars. As it was pointed out earlier in the paper more research is needed to relate all the variables involved and it would be probably hasty to draw any definite conclusions about the possible results of such research project.