HYSTERETIC BEHAVIOR OF CONCRETE FLEXURAL MEMBERS REINFORCED WITH HIGH STRENGTH STEEL

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SUMMARY

Different aspects of the behavior of concrete members reinforced with high strength steels are being studied in a research program at the Institute of Engineering of the National University of Mexico. As a part of this program seven flexural elements were tested under large displacement reversals to study the hysteretic behavior for different types and amounts of longitudinal reinforcement. Yielding and unyielding steels were used whose specified yield stress was either 420 or 600 MPa.

Comparison of the behavior of different speciments indicates that when unyielding steel is used, the amount of energy dissipated for a given displacement level is significantly lower than for steel with a large yield plateau. Hysteresis loops for the first case also show greater deterioration under repetitions of cycles for large displacements. Progressive bond slip due to the large stresses that must be transferred to the concrete seems to be mostly responsible for this unfavorable behavior.

INTRODUCTION

The main advantages of using high strength steel as concrete reinforcement are based on the significant savings that can be obtained due to the fact that, whereas cost of steel increases only marginally for higher yield stresses, the amount of reinforcement needed to reach a given strength is in many cases inversely proportional to the yield stress, f_y.

Making use of this advantage, reinforcement of grade 300(f_y = 300 MPa) or lower, has progresively disappeared from the market in Mexico and has been substituted by grade 420 (f_y = 420 MPa) steel that presently constitutes about 90% of the total production of reinforcing bars.

Since about 25 years ago grade 600 (f_y = 600 MPa) bars (cold twisted TOR type) have been introduced in Mexico, where although it constitutes only a small percentage of total production (around 6%) it is frequently used in rather important structures such as medium to high rise concrete buildings. Though there is considerable interest among the steel industry to increase the production of high strength steel, its use is strongly

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opposed by a large number of structural engineers.

Objections to the use of grade 600 steel are related to deflections and cracking of flexural members under service loads, to the possibility of local damages when bars are bent with small curvature radii as in stirrups, to bond and anchorage problems and to the effectiveness of stirrup contribution to shear strength. It has been demostrated that most of these objections can be overcome with proper precautions in design and construction. Major objections still subsist related to seismic behavior of concrete structures reinforced with those types of steel that lack a yield plateau and have lower strain at rupture, thus giving rise to structures with less ductility and lower energy dissipation capacity.

Most recommendations for earthquake resistant design of concrete structures do not cover, or explicitly forbid, the use of high strength bars. Mexican codes allow the use of grade 600 steel bars if structures are designed for seismic forces larger than those specified for common steel, thus reducing the ductility demand under the design earthquake. Code recommendations seem to be based on general considerations concerning the effect of lower steel ductility, whereas experimental evidence with respect to the behavior of concrete elements reinforced with this type of steel is lacking.

As a part of a research program aiming at establishing design criteria for concrete structures reinforced with high strength steel, an experimental study was carried out to compare the behavior under alternating loads of flexural members reinforced with different amounts of grade 420 and grade 600 steel. Main findings are reported in this paper. Previous stages of the research dealt with the effectiveness of stirrups, with the effects of bar bending during fabrication and with the behavior as confinement steel in compression members.

EXPERIMENTAL STUDY

Typical stress-strain curves of reinforcing bars produced in Mexico are shown in fig 1. Grade 420 bars must show, supposedly, a definite yielding plateau, as in curve a) of fig 1; nevertheless, frequently, and specially for small diameters, curves that do not exhibit yielding are found, as in curve b). Strain at rupture consistently exceeds 20% and actual yield stress commonly exceeds specified yield stress by about 25% in average. Ratio of maximum to yield stress is around 1.5. Grade 600 bars obtained by cold twisting bars that otherwise would be grade 420, never show a proper yielding and commonly reach a unit strain of around 15% at rupture. Actual yield stress is in average only 10% higher than the specified minimum and ratio of maximum to yield stress is slightly higher than 1.15.

Seven specimens as shown in fig 2 were built simulating the end of a flexural member and its connection to a column. The horizontal member, representing the column, was fixed to the load floor and was identical for all specimens. In the vertical member, representing the beam, both the grade and the amount of longitudinal reinforcement were varied. Pairs of specimens were formed, identical except for the quality of the reinforcement that was grade 600 in one and grade 420 with definite yield in the other. The area of the main longitudinal reinforcement was chosen to represent, approximately, 20, 40 and 60% of the balanced ratio. Reinforcement area in the other side of the section was one half the main reinforcement. For the lower amount of reinforcement a third specimen was tested reinforced with grade 420 bars that did not show a yield plateau. Stirrups spaced at half depth were designed in order to reach a shear strength at least 30% higher than flexural strength.

Reinforcement met the requirements of Mexican codes when design seismic forces corresponding to linear behavior are reduced by a factor of four to take advantage of energy dissipation capacity in the inelastic range. A maximum reduction factor of six is allowed by the codes but related to much more stringent ductility requirements and only for grade 420 bars.

Specimens were subjected to cycles of alternating lateral loads up to the same maximum deflection in each direction. At least three cycles were repeated for each deformation level; afterwards, the level of maximum displacement was increased. Lateral displacements, strains in longitudinal and transverse reinforcement and rotations of two sections near the end of the beam were measured as indicated in fig 3.

Observed behavior can be summarized as follows. Load-displacement curves were approximately linear when reinforcement was maintened in the elastic range. Nevertheless some area was enclosed in the hysteresis loops even for very low load levels. When the yield stress of reinforcement was exceeded, hysteresis loops took a characteristic shape with an initial branch of low stiffness followed by a zone of higher slope that continued until the maximum displacement of the cycle. The low stiffness branch was larger for specimens reinforced with grade 600 bars than for those reinforced with grade 420 steel with a definite yielding, thus giving rise to a lower area enclosed by the hysteresis loops in the former case (fig 4). The behavior of the specimen reinforced with grade 42 bars that did not show a clear yielding was very much like that of specimens reinforced with grade 600 bars (fig 5).

For displacement levels not much exceeding first yielding, hysteresis loops were clearly stable for all the types of steel, but when maximum displacement largely exceeded yield, a progressive degradation was observed: both the load corresponding to a given displacement and the area enclosed by the loops decreased continuously, though if very slightly, from one cycle to the following. Failure was due to crushing of the concrete when load was applied in the stronger direction. The repetition of load caused a progressive increase in the depth of the crushed concrete, followed by buckling of the compression reinforcement that originated a marked reduction of load capacity.

Main data and results are summarized in table 1. An acceptable agreement between calculated and resisting moments in each direction was found. Each specimen of a pair was designed for the same yielding moment; due to the higher ration between maximum and yield stress for grade 420 than for grade 600 bars, maximum resisting moments are larger in the former case.

Ductility factors measured on the envelope of the hysteresis loops are in all cases greater than four; they decrease for larger reinforcement ratios and are only slightly lower for grade 600 than for grade 420 bars. In specimen AR-1 ductility could not be determined because failure was due to tensile fracture of reinforcing bars in the weak direction. In this single case the amount of reinforcement was very low and special small diameter cold worked bars were used whose strain at failure was about 4%, that is much lower than the limit allowed by the standards.

Energy dissipation capacity was measured by indexes EA_1 and EA_2 of table 1. They represent the ratio between the area enclosed by the hysteresis loops and the area that would correspond to a perfectly elastoplastic behavior for the same initial stiffness, resisting moment and maximum deformation (fig 6). EA_1 represent the situation for displacement not much exceeding yielding, whereas EA_2 is measured in cycles near failure. In all cases both indexes are much lower than one, indicating that the amount of energy that can be dissipated is much lower than that corresponding to an elastoplastic behavior. Also indexes for grade 420 bars with definite yielding are always at least 50% larger than those for unyielding bars.

COMMENTS ON THE EXPERIMENTAL RESULTS

Main features of the observed behavior can be explained as follows.

When the amount of longitudinal reinforcement meets the limits allowed by the codes, failure of flexural members is governed by the crushing of concrete in compression, the tensile strain of reinforcement being much lower than that causing rupture. Compression reinforcement cannot delay the drop in load capacity unless it is restricted against buckling by very closely spaced ties. The latter was not the case for the specimens tested, thus the mode of failure was not influenced by the type of reinforcing steel except for specimen AR-1 where an exceptionally brittle type of steel was used. In any case, the strength and the form of the envelope curve of the hysteresis loops could be rather closely predicted from general principles of concrete behavior under bending and axial load.

Ductility factors were in all cases higher than four. It must be pointed out that their value is very much related to the way of defining the yield strain and the initial slope of the load-displacement curve. Much higher values can be obtained if different criteria for defining these parameters are used. It is more significant and reliable to speak in terms of maximum rotation of the beam (maximum displacement at the free end divided by the length of the beam). This maximum rotation was in the range of 3 to 4%, except for specimen AR-1 due to the exceptionally brittle steel used. Much larger rotation capacity could be attained if closely spaced transverse reinforcement was placed to prevent or to delay buckling of compression reinforcement. It must also be noted that the ductility factors obtained in the tests are rather lower than those that can be expected in tests under monotonic loads, because the repetition of load causes a faster deterioration of strength once the crushing of concrete in compression has initiated.

The shape of the hysteresis loops can be explained as follows. When the load is applied in one direction, cracks open in the tension side; in the unloading stage these cracks do not close completely and some load level in the opposite direction is needed to close the cracks; while this takes place the member stiffness is low because there is practically no contact between concrete on both sided of the crack in the compresion zones. Before the steel yields, the effect of this phenomenon can be disregarded because the crack width is small. When yield is attained, progressively increasing residual strains are left in the bars after the unloading and, mainly, there is some loss of bond between steel and concrete on both sides of the cracks. Bars are subjected to high stresses in compression in one half of the loading cycle and to large post yielding tensile strains in the following half of the cycle. Concrete in contact with bar deformations is crushed locally and allows some sliding of the bar. In the length where bond is lost, the stress in the bars is constant and equals the stress in the section of maximum moment. Due to the repetition of load this length increases progressively, the residual width of the crack after unloading also increases and so does the length of the initial branch of low stiffness. Once the cracks close the member recovers stiffness.

Measurements taken during the tests indicated that in the post yielding range the main cause of lateral displacement was the rotation at the base of the member due to the opening of a large flexural crack, whose width was mainly originated by the loss of bond in the anchorage zone, that penetrated more deeply in the column after each repetion of load (fig 7). The residual crack width after unloading causes the low stiffness of the initial branch.

Shear cracking has been found in other test to be responsible for a similar hysteretic behavior; nevertheless in our tests the width of shear cracks was small and it is considered that this did not significantly contributed to the behavior.

The kind of behavior just described was observed both in specimens reinforced with bars with definite yielding and in those with unyielding bars. In the second case the problem of loss of bond was more severe because the stress in the bars increases continuously in the inelastic range and the transmission of bond stresses to the concrete is more critical. If steel yields, bond stresses are constant in a zone of increasing rotations that can give rise to an higher energy dissipation capacity.

CONCLUSIONS REGARDING THE SEISMIC BEHAVIOR

Two aspects of the stress-strain curve of high strength bars significantly affect the seismic behavior of reinforced concrete structures: the lack of a yield plateau and the high stress transmitted to the concrete.

Without a yield plateau there is no possibility for the formation of a proper plastic hinge where large rotations can take place without changes of the internal forces in the member and where large amounts of energy can be dissipated by nearly elasto-plastic hysteresis loops. In this case inelastic rotations are associated to continuously increasing bending moments generating hysteresis loops with lower area enclosed and allowing other internal forces to increase above values considered in the design, thus giving rise to the possibility of other modes of failure of a non ductile nature.

On the other hand, high bond stresses under load levels near yielding or lower, originate the sliding of bars and a low stiffness branch in the load-displacement curve that drastically reduces the area enclosed in the hysteresis loops. Moreover, as the bond failure is of a degrading nature, the length of the low stiffness branch increases continuously, though slightly, producing a progressive deterioration of the energy dissipation capacity.

Step by step dynamic inelastic analyses of one degree of freedom systems subjected to different hystories of ground movements have shown that the ductility demand of systems with stable hysteresis loops is not much affected by reductions of the area enclosed by loops in the range of 30 to 100% of that corresponding to an elasto-plastic behaviero. Therefore, it must be expected that the lower energy dissipation capacity that will corrrespond to concrete structures reinforced with high strenghth bars would not significantly impair their seismic behavior with respect to these reinforced with grade 420 bars, as the ductility is almost the same.

What seems to be of more concern is the degradation due to the loss of bond that would deteriorate the energy dissipation capacity and which, as has been demonstrated in other studies, is practically impossible to restore, in such a way that a structure that has been subjected to large inelastic deformations during an earthquakes would be affected in its ability to resist future shakes.

These shortcomings of the seismic behavior are not exclusive of high strength bars; grade 420 bars without definite yielding present the same type of behavior. Considering that the present standards do not require grade 420 bars to yield, it is considered that the same restrictions must be imposed to both types of reinforcement.

Much larger rotations can be obtained in flexural members reinforced with both types of bars, if compression reinforcement is properly tied by closely spaced stirrups in order to avoid buckling and to confine the concrete core; nevertheless these higher rotations would be associated to higher stresses in the tensile steel and to more severe bond problems that would impair the energy dissipation capacity due to the progressive deterioration of strength and stiffness. Therefore it is considered that for these types of reinforcement if is preferable no to make use of a large rotation capacity and to design the structure for seismic actions that are only slightly reduced for ductility.

It must be pointed out that the experimental evidence obtained is limited to concrete members reinforced with bars of rather small diameters (#6 or lower) and that the problems of bond deterioration could be more severe for larger bars. Bond tests of TOR type bars have shown that the efficiency of their deformations decreases in bars of large diameters and that bond stresses that can be developed for #8 bars are, for instance, 25% lower than those developed by #4 bars.

A further stage of the research program is being undertaken where specimens of a larger size reinforced with bars of a greater diameter are tested.

TABLE 1. PROPERTIES AND RESULTS OF TESTS

Specimen	f'_c kg/cm^2		A_s, A'_s	f_y kg/cm^2	ρ/ρ_b	Stirrups d/2	Max Load kN	Ductility factor		
D-1	32	Strong Weak	3#3 2#2.5	460 490	0.22	#2	40 19	8	0.19	0.20
DD-1	31	Strong Weak	3#3 2#2.5	430 390	0.20	#2	31 16	8.6	0.29	0.33
AR-1	33	Strong Weak	2#3 2#2	620 610	0.22	#1.5	34 12	>5.0	0.17	—
D-3	29	Strong Weak	3#4 3#3	470 520	0.36	#2	55 42	5.4	0.27	0.33
AR-2	29	Strong Weak	2#4 2#3	630 620	0.39	#3	53 28	4.5	0.10	0.21
D-3	28	Strong Weak	2#6 2#4	500 490	0.60	#2.5	88 47	4.5	0.22	0.29
AR-3	34	Strong Weak	2#5 3#3	620 620	0.58	#2	84 44	4.1	0.12	0.21

$$\rho = \frac{A_s}{bd}\frac{f_y}{f'_c} \;;\; \frac{\rho}{\rho_b} = \frac{\rho}{\frac{0.68 f'_c}{f_y}\,\frac{6000}{6000+f_y} + \rho'\frac{f_y}{f'_c}}$$

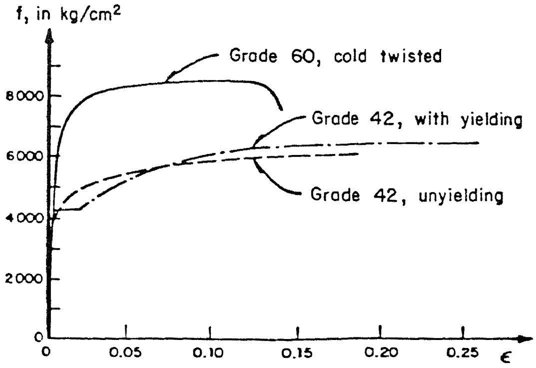

Fig 1. Stress-strain curves of different types of reinforcement

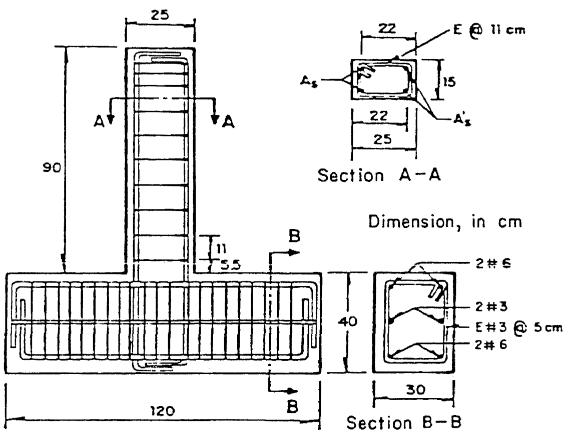

Fig 2. Specimens' characteristics

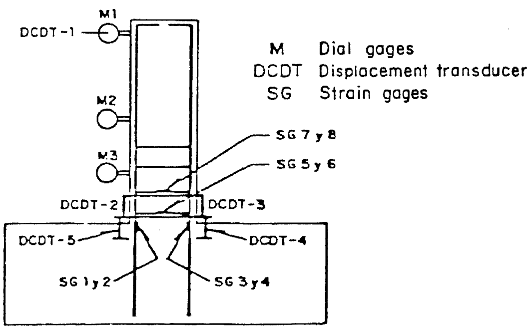

Fig 3. Instrumentation

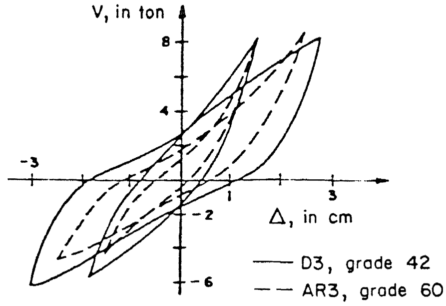

Fig 4. Selected hysteresis loops for specimens reinforced with grade 42 and grade 60 bars

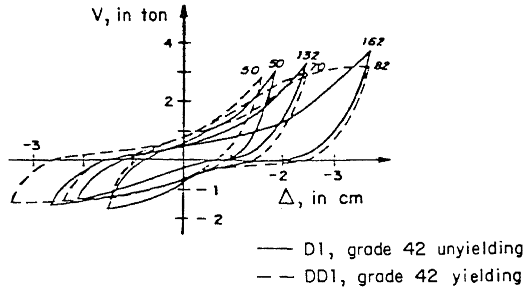

Fig 5. Selected hysteresis loops for specimens reinforced with two different types of grade 42 bars

A_H = Area of experimental hysteresis loop

A_{EP} = Area of corresponding elastoplastic loop (inside dotted lines)

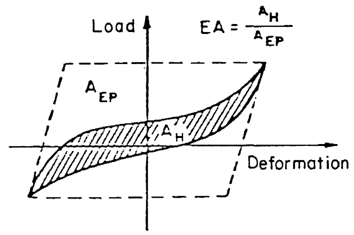

Fig 6. Definition of energy dissipation index

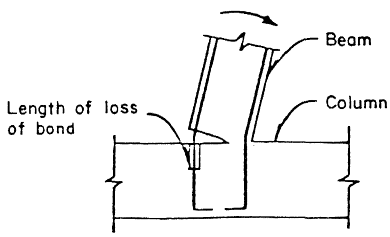

Fig 7. Mechanism that originate deterioration for loss of bond in the anchorage length