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CYCLIC RESPONSE OF CONCRETE COLUMNS REINFORCED WITH HIGH-STRENGTH STEEL

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ABSTRACT

In this paper the use high-strength bars (having a yield stress exceeding 80 ksi) as longitudinal reinforcement for columns is reconsidered. The focus of the work presented is on columns meeting current requirements for columns of special moment frames and columns with axial loads not exceeding the axial load at balance. The flexural strength of these columns is controlled by the strength of the steel. It follows that two sections with different grades of steel have similar moment capacities as long as the product of reinforcement ratio and yield stress is similar for both sections.

Experimental and analytical tests of this hypothesis are presented. Tests of columns reinforced with either 60-ksi or 120-ksi steel were conducted. It is shown that both types of columns can be used to reach drift ratios of up to 4%. These tests also show that columns reinforced with A1035 120-ksi steel reinforcement, which does not exhibit a yield plateau, have smaller drift capacities than columns reinforced with A706 60-ksi steel with a yield plateau. The difference in drift capacity is attributed to the differences in the shape of the stress-strain curve, which leads to differences in the distribution of curvature.

Introduction

For decades, the construction industry has used steel reinforcement for concrete elements with nominal yield stresses not exceeding 60 ksi. By increasing the yield stress of the reinforcing steel, a number of benefits could be achieved. A few of the possible advantages are the following:

1. Reduction in required amount of reinforcing
2. Reduction in shipping costs
3. Reduction in labor costs
4. Reduction in environmental impact

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5. Reduction in delays during construction

The above advantages can only be achieved if a concrete element reinforced with high-strength steel behaves similarly to a concrete element reinforced with grade-60 steel. There is, however, a long list of criteria on which the behavior can be judged. A few of the criteria discussed in this paper are strength, deformation capacity, crack widths at service loads, and energy dissipation in the case of dynamic response.

In order to take advantage of a higher yield stress in the reinforcing steel, higher stresses in the steel at service level loads must be expected. With higher tensile stresses in the steel, wider (or more) cracks can also be expected. To avoid this, columns designed as part of a seismic force resisting system are the focus of this investigation. The issue of crack width becomes less of a concern in columns because the axial load closes the cracks. In addition, when a column is designed as part of a seismic force resisting system, the service level moments are typically small as compared with the moment capacity. This allows for a small (or non-existent) tensile stress in the reinforcing steel during service. Only in the event of an earthquake would cracks in the column open up, and in that scenario, cracks are rarely of concern.

Moment-Curvature Analysis

New technology has allowed metallurgists to create steels with high strength and high ductility. An ASTM specification introduced in 2007 describes a type of steel with a yield stress of 120 ksi (determined using the 0.2% strain offset method) and an elongation at rupture of at least 7% (ASTM A1035 2007). This steel forms the basis of our comparisons with traditional grade-60 reinforcement (ASTM A706 2007). A plot of sample stress-strain curves of these two steel grades can be seen in Figure 1.

At first glance, it seems reasonable to expect that if one uses, for example, 3.0% reinforcement ratio of grade-60 reinforcement in a column, the same moment capacity could be achieved using 1.5% reinforcement ratio of grade-120 reinforcement in the column. Like so many other things, however, the devil is in the details. The overall shape of the high-strength stress-strain curve is different than that of traditional steel. As such, a qualitative difference should be expected in the moment-curvature response of columns using high-strength steel.

To see how the different steel material properties translated into sectional behavior, moment-curvature analyses were run on comparable sections – having approximately the same moment capacities, but different amounts and strengths of steel –. Each section is made from 5000 psi concrete which is assumed to follow the expression proposed by Hognestad to relate stress and strain for concrete in compression (Hognestad, et. al 1955). Each has an eighteen-inch-square cross section with two and a half inches of cover to the center of the steel at each face. An axial load corresponding to 0.20 times the product of the gross cross-sectional area and the concrete compressive strength is assumed to be applied to each column.

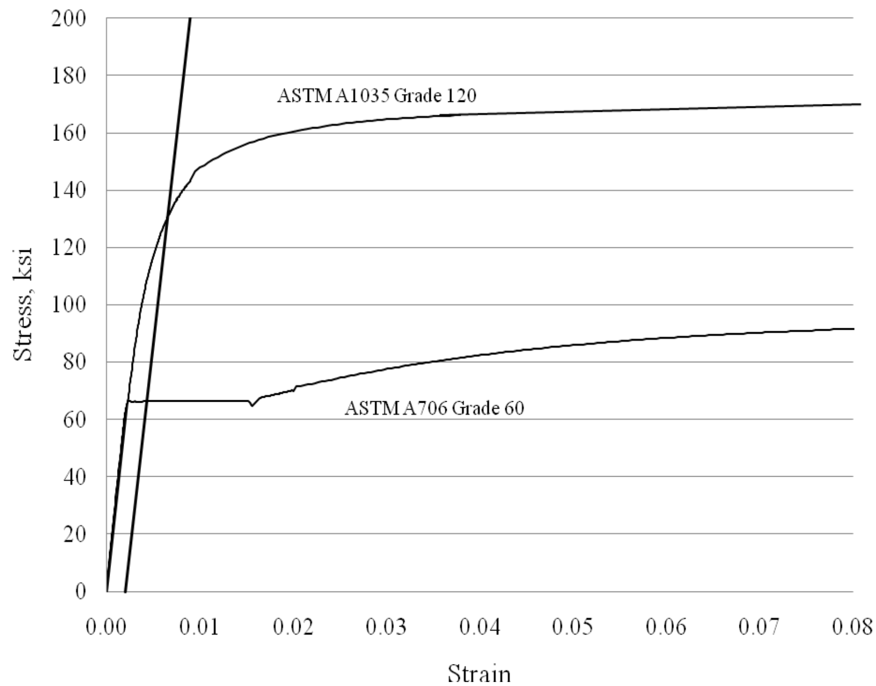


Figure 1. Stress-strain relationship of sample high-strength and traditional reinforcing bars

The difference in the sections is in the amount and grade of steel. The first is reinforced with grade-60 reinforcement amounting to a 3.0% reinforcement ratio based on the gross cross sectional area. The other is reinforced with grade-120 steel amounting to a 1.5% reinforcement ratio. The grade-60 steel is assumed to have an elastic, plastic, then linear strain hardening stress-strain relationship while the grade-120 steel is assumed to fit a line given by an equation proposed by Menegotto and Pinto (Menegotto and Pinto 1973). The constants in the equation were adjusted so that the line would fit a set of stress-strain data obtained by the authors.

The comparison between the two moment-curvature responses can be seen in Figure 2. Similar curves can be made for columns of different cross sections, reinforcement ratios, axial loads, and concrete compressive strengths. As expected, the two plots are similar before the concrete cracks, after which, the section with traditional reinforcement is stiffer. This is because although the high-strength steel is stronger, both steels have the same elastic modulus. The stiffness of the column is not a function of the strength of the steel, rather of how much steel is provided. Both sections eventually reach about the same moment capacity; however, the section reinforced with high-strength steel requires a higher curvature than the traditionally-reinforced section.

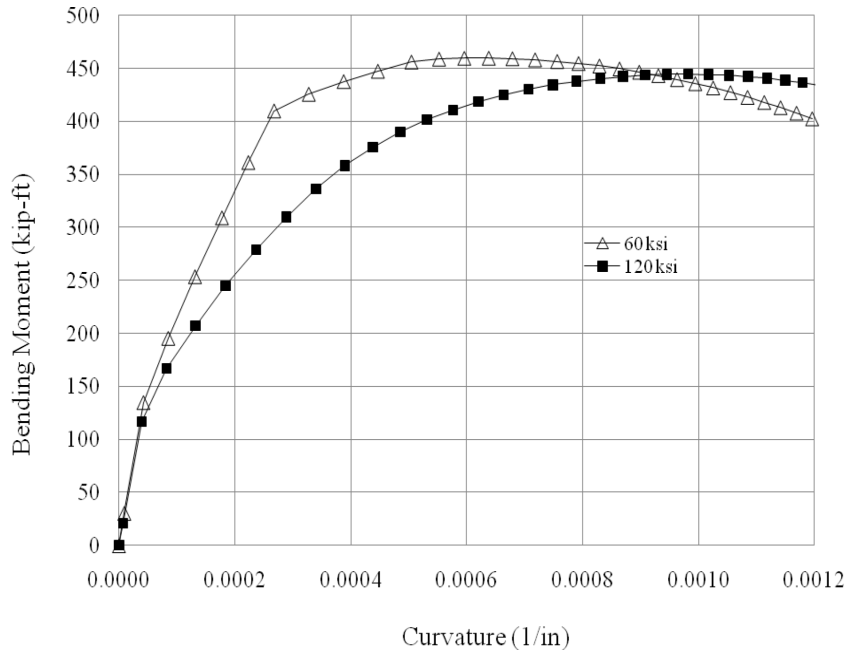


Figure 2. Moment-curvature diagrams of sample columns reinforced with traditional and high-strength steels

Experimental Study

Although the moment-curvature analysis provides insight, it is difficult to translate sectional behavior into the global behavior of a column. Therefore, a series of tests were conducted on columns reinforced with high-strength steel. Of highest interest to the authors was the behavior of these columns when subjected to cyclic displacement reversals. From these data, the effect of an earthquake on a structure made of this steel could be inferred. A matrix of the test program can be seen in Table 1 and a drawing of the specimens to be tested can be seen in Figure 3.

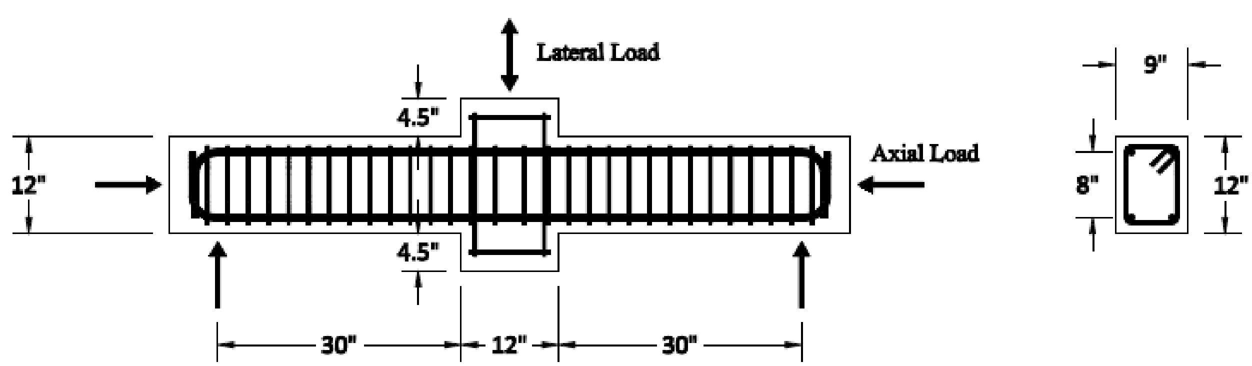


Figure 3. Drawing of typical test specimen

Table 1. Test matrix of relevant specimens

No.	Designation	f_y	A_s / A_g	$P / (f'_c A_g)$
		ksi	%	
1	CC-3.3-10	60	3.3	0.10
2	UC-1.6-10	120	1.6	0.10
8	CC-2.4-20	60	2.4	0.20
9	UC-1.1-20	120	1.1	0.20

Where: f_y = yield stress of longitudinal reinforcement,
A_s = total cross-sectional area of longitudinal reinforcement,
A_g = gross cross-sectional area of column,
P = applied axial load, and
f'_c = compressive strength of concrete.

The two important comparisons to make in this series of tests are between specimens 1 and 2, and between specimens 8 and 9. In these two sets, all variables were held constant except for the strength and amount of reinforcement provided, and the applied axial load. The longitudinal reinforcement in the column pairs was proportioned such that the moment capacities would be nearly equal. Specifically, specimens 2 and 9 had about half the amount of steel as specimens 1 and 8, respectively. The other variable considered was the applied axial load. In specimens 1 and 2, the applied axial load was $0.10\, f'_c\, A_g$, while it was twice that in specimens 8 and 9. The sizes of longitudinal reinforcing bars ranged from #5 to #7 in the four columns described. For all columns, shear and confinement reinforcement was provided by #3 bars spaced at 2.5 inches.

Shear-Drift Behavior

The shear-drift curves from the four tests described can be seen in Figures 4 through 7. Only the plot for the controlling half-column is shown.

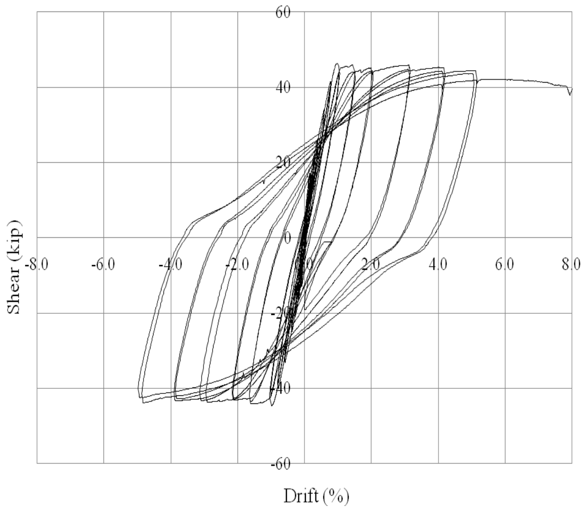


Figure 4. Shear-drift plot for specimen 1 (CC-3.3-10)

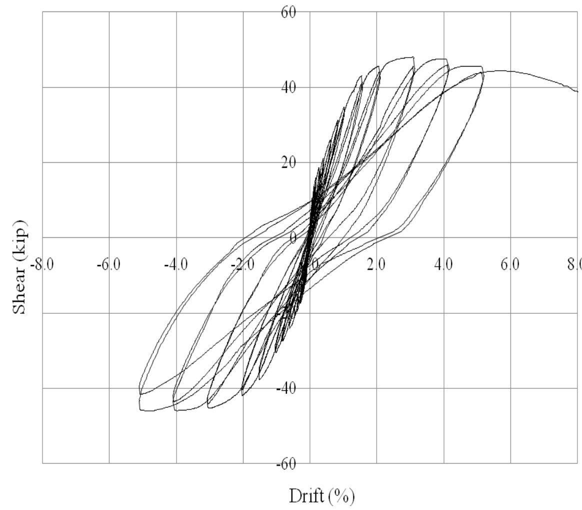


Figure 5. Shear-drift plot for specimen 2 (UC-1.6-10)

First, a comparison between specimens 1 and 2 will be made. As was expected, the moment capacity of both columns was approximately the same even though the column reinforced with high-strength reinforcement had about half the amount of steel. Another thing to notice is that, as the moment-curvature relationship would suggest, specimen 2 was more flexible after cracking. Both columns exhibited large deformation capacities – exceeding 8% drift ratio-. However, at the end of the response, it can be seen that specimen 2 was losing resistance at a faster rate than specimen 1.

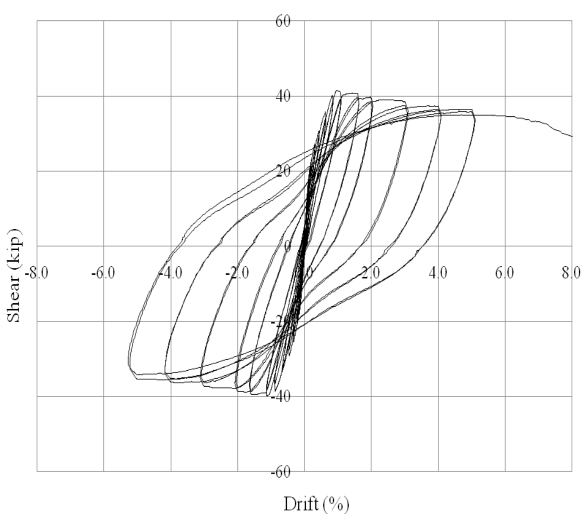


Figure 6. Shear-drift plot for specimen 8 (CC-2.4-20)

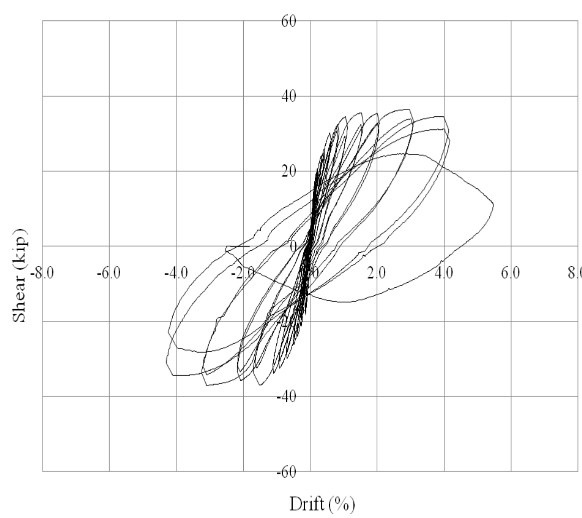


Figure 7. Shear-drift plot for specimen 9 (UC-1.1-20)

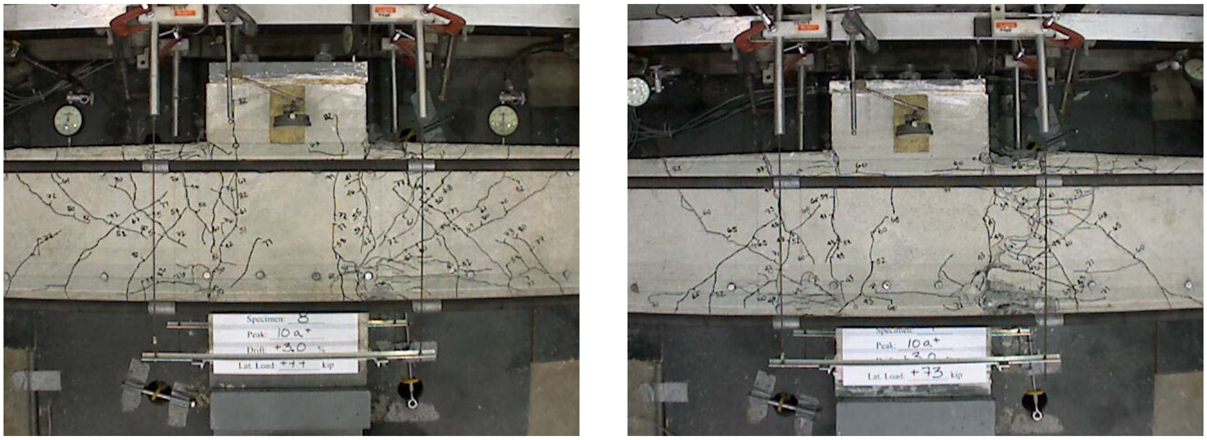
Some of the same trends seen in the comparison between specimens 1 and 2 can be seen when comparing specimens 8 and 9. In this comparison, the trends become more apparent. This is because the axial load is twice as high, which, in a way, amplifies the negative aspects of both columns. The moment capacities were nearly the same in the two specimens, except the one using high-strength reinforcement, specimen 9, had a moment capacity about 10% lower than specimen 8. The difference in strength of the columns was nearly proportional to the difference in the product of the reinforcement ratio and the yield stress of the steel. This suggests that the method used to compute the moment-curvature relationship holds for sections reinforced with high-strength steel.

Specimen 9 (high-strength reinforcement) failed at a drift ratio of about 4% while specimen 8 (traditional reinforcement) failed at a drift ratio exceeding 7%. In both cases, the failure mode was buckling of longitudinal reinforcement, even though the transverse hoops were spaced at a quarter of the effective depth of the column (2.5 inches). The difference, however, is the method in which the bars buckled. There was a sliding shear failure in specimen 8 which led to all four longitudinal bars buckling in the same direction. In specimen 9, however, the bars in compression buckled first, causing the reduction in load carrying capacity. The culprit for the failure of specimen 9 at a lower drift ratio than specimen 8 is explored in the following section.

Curvature Distribution

The deflection in a column is not just a function of the strain at the section of maximum moment. It is also a function of, among other things, distribution of curvature along the length of the column. The curvature distribution is not yet completely understood, but the shape of the stress-strain curve of the reinforcement is an important factor. Because the stress-strain behavior of high-strength steel is different from that of traditional steel, it would not be unreasonable to expect that the curvature distribution of columns reinforced with different steels is different. This can best be seen when comparing specimens 8 and 9.

Figure 8 shows photographs of specimens 8 and 9 at a drift ratio of 3% at the peak positive displacement. The columns at other drift ratios exhibited a similar pattern. There is a clear qualitative difference between the two crack patterns. In specimen 9, there was more spalling and were more splitting cracks. In addition, the cracking in specimen 8 extended further along the half-column than they did in specimen 9. From the cracking pattern, we can conclude that the curvature distribution was different between the two columns.



Specimen 8 (CC-2.4-20) Specimen 9 (UC-1.1-20)

Figure 8. Crack pattern of specimens 8 and 9 at 3.0% drift ratio

Because of the shorter distance over which cracking was spread in specimen 9, the portion of the column in the region with cracks had to undergo a higher average curvature than the cracked regions of the column reinforced with traditional steel. This higher average curvature is the only way that the columns could achieve the same deflection. The increase in average curvature leads to higher strain gradients through the column, which causes the failure of the concrete earlier. When crushing of the concrete progresses into the core of the column, more of the necessary compression force is transferred to the reinforcing bars, leading to buckling of the compression reinforcement. A picture of specimen 9 after failure and after the loose concrete was removed can be seen in Figure 9.

Figure 9. Specimen 9 after testing and after the loose concrete was removed

Hysteretic Response

Another clear difference between the specimens reinforced with traditional steel and those reinforced with high-strength steel is the hysteretic energy dissipation. The cause of this is the stiffness of the columns. Because about half the steel is used in specimens 2 and 9, the stiffnesses of specimens 2 and 9 were about half those of specimens 1 and 9, respectively. Because the specimens reinforced with high-strength steel were less stiff, the area contained inside the hysteretic loops is smaller. Therefore, they will dissipate less energy due to hysteresis.

If a structure were to be built using high-strength steel technology, it is likely that the columns would use high-strength steel while the beams would use traditional-strength reinforcement. Therefore, the global pushover curve would resemble something between the two stiffness extremes. More studies must be conducted as to how the local hysteretic properties of a column would affect the global behavior of a structure if subjected to a seismic loading event.

Summary and Conclusions

Tests of columns under cyclic load reversals show that columns reinforced with A1035 120-ksi steel reinforcement can reach drift ratios of 4%. These tests also show that columns reinforced with A1035 120-ksi steel reinforcement which does not exhibit a yield plateau have smaller drift capacities than columns reinforced with (twice as much) A706 60-ksi steel reinforcement with a yield plateau. The difference in drift capacity is attributed to the differences in the shape of the stress-strain curves, which leads to differences in the distribution of curvature.

References

ASTM A1035, "Standard Specification for Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement," ASTM International, West Conshohockem, PA, 2007, 5 pp.

ASTM A706, "Standard Specification for Low-Alloy Deformed and Plain Bars for Concrete Reinforcement," ASTM International, West Conshohockem, PA, 2007, 6 pp.

Hognestad, E., Hanson, N., and McHenry, D., "Concrete Stress Distribution in Ultimate Strength Design," *ACI Journal, Proceedings,* Vol. 52, No. 4, December 1955, pp. 475-479.

Menegotto, M., and Pinto, P. E., "Method of Analysis for Cyclically Loaded R.C. Plane Frames Including Changes in Geometry and Non-Elastic Behavior of Elements under Combined Normal Force and Bending," Preliminary Report, International Association for Bridge and Structural Engineering Symposium, Lisbon, Portugal, IABSE Reports, Vol. 13, 1973, pp. 15-22.