

Member ductility experiment with high strength steel and concrete

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ABSTRACT: The results of an experimental program on simply supported beams subjected to two point loading are presented. Nine specimens, with three different reinforcement ratio (equal to $p = 0.007$, 0.009 and 0.0128) and three different concrete strengths (equal to $f'_c = 5.25$ ksi, 6.5 ksi and 7.25 ksi) have been tested. An attempt is made to classify the performance of the specimens according to the ductility they exhibited varying tensile steel ratio and concrete compressive strength. Moment-curvature relation, depth of neutral axis, strain in the materials are considered as performance criteria in this study. The concept of minimum flexural ductility was also considered in this paper.

1 INTRODUCTION

An attempt is made to classify the performance of high strength steel specimens according to the ductility they exhibited varying tensile steel ratio and concrete compressive strength. Moment-curvature relation, depth of neutral axis, strain in the materials are considered as performance criteria in this study. Higher grades are often used to permit smaller concrete members, relating to the space problems for placement of the reinforcement. Even though the steel ordinarily constitutes only a few percent of the total volume of reinforced concrete, it is a major cost factor. This reduction in concrete member and percentage of steel tend to reduce the flexure stiffness and ductility of a member. Flexural strength and stiffness can be easily evaluated using the ordinary beam bending theory, but there exists no simple method for evaluating the flexural ductility of a reinforced concrete (RC) beam. It is generally considered that in the interests of safety, it is essential to provide a certain minimum level of flexural ductility and that for this purpose, just designing the beam sections to be under-reinforced is not sufficient. In most of the existing design codes, reinforcement detailing rules, which impose limits on either the tension steel ratio or the neutral axis depth, have been incorporated to guarantee the provision of minimum flexural ductility.

2 RESEARCH SIGNIFICANCE

This research would help to understand the ductility parameters. This would also provide information on role of concrete strength and Grade 500 steel for beam on moment-curvature relation, ductility ratio and neutral axis depth.

3 TEST SPECIMENS

The beams taken for the experiment were 7.5 feet c/c long with a 10 inch \times 12 inch rectangular cross section and clear span was 7ft and 9 inch. These beams were simply supported and subjected to a symmetric loading composed of two equal concentrated forces. Such loading led, in theory, to pure bending between applied forces. The failure in the mentioned area between applied forces occurred always by simple bending. A total of nine rectangular concrete beams were fabricated and tested. Experimental parameters are shown in Table 1.

Table 1. Experimental parameters

| Steel ratio | Compressive strength | Designation |
|-------------|----------------------|-------------|
| 0.0128 | 6.5 | B1 |
| | 5.25 | B4 |
| | 7.25 | B7 |
| 0.009 | 6.5 | B2 |
| | 5.25 | B5 |
| | 7.25 | B8 |
| 0.007 | 6.5 | B3 |
| | 5.25 | B6 |
| | 7.25 | B9 |

In order to assure a failure by flexure located between the point loads, a sufficient amount of stirrups was put in the zone outside the point loads in order to prevent failure by shear. The central zone between point loads had no stirrups to avoid confinement of the concrete. Since this zone was, in theory, in pure bending, the stirrups would not be necessary as far as the resistance is concerned.

4 REINFORCEMENT

Out of nine, three beams were reinforced with 3 #6 Grade 500 steel rebars, other three beams were reinforced with 2#6 Grade 500 steel rebars and rest three beams were reinforced with 2#5 Grade 500 steel rebars in the tension side. The clear cover was 1 inch. Each beam was provided with 2#3 top bars for holding the stirrup. All beams were reinforced against shear failure by placing #3 Grade 500 steel at a spacing of 5 in centre to centre closed-type stirrups for 5.25 and 6.5 ksi concrete and 3 in centre to centre for 7.25 ksi concrete. In Figure 1 description of all beams are given concisely.

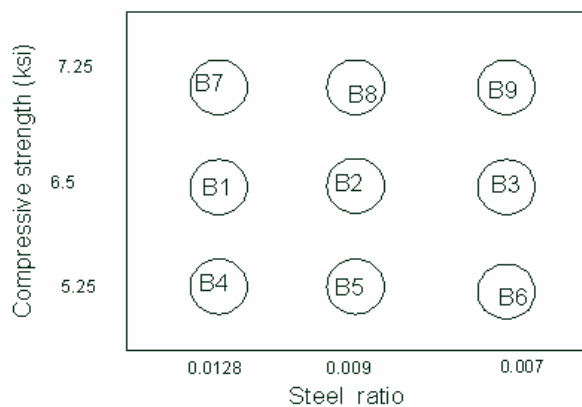


Figure 1. Beam specimens with different steel ratio and concrete strength

5 EXPERIMENTAL PROGRAM

Portland Composite Cement (PCC) conforming to BDS-EN 197 Part 1 was used for all mixes. Sylhet sand with specific gravity of 2.68 was used as fine aggregates. Crushed stone with specific gravity of 2.69 was used as coarse aggregates. Grade 500 steel was used as reinforcing steel. Experimental design was prepared to understand the non-linear behavior of beam under monotonic loading for different steel ratios and concrete compressive strengths. Three steel ratios and three concrete compressive strengths were taken, which is shown in Figure 1 and also given in Table 1. The compressive strength of the concrete was determined by

compression tests on typical samples of each mix of concrete. The type of mix design used to produce the concrete and the details of each mix are presented in Table 2.

Table 2. Components and amounts for mix design

| Component | Mix design (kg/m ³) | | |
|-------------------|---------------------------------|-------|-------|
| | 5 ksi | 6 ksi | 7 ksi |
| Cement | 435 | 480 | 520 |
| water | 200 | 192 | 208 |
| Sylhet sand | 710 | 730 | 668 |
| Crushed stone | 1064 | 1094 | 1006 |
| FA/TA | 0.4 | 0.4 | 0.4 |
| W/C | 0.46 | 0.4 | 0.4 |
| Density | 2408 | 2412 | 2400 |
| Achieved strength | 5.25 | 6.5 | 7.25 |

6 LOADING PROCEDURE

Concrete beams were simply supported and two point loading was applied. Monotonic loading was applied till the deflection of the beam starts to flow. The load was applied at 3 feet distance from the end of each beam. Load was applied in tons. Load was increasing by about 2 tons and cracking patterns were observed and recorded by camera. The experimental setup is shown in Figure 2. One linear voltage displacement transducers (LVDT) were placed at centre of the beam to measure the deflections at centre. The crack propagations were monitored using hand held microscope. Five holes were made in along the depth of the beam to measure the strain in the cross section. All strains, crack propagation and deflection measurements were measured at every load increment. The first crack load was noted immediately after the formation and all the cracks were marked as and when they propagated in the beam. Propagation of cracks can be seen in the Figure 2.



Figure 2. Beam specimen in test frame

7 RESULTS

7.1 Strain in cross section

The tensile and compressive strains of reinforcement and concrete respectively were measured at every load increment. The negative values show the compressive strains in the concrete, while the tensile strains in the reinforcement are shown in positive values. The strains were linear in beams until yielding of steel and then rapidly increased before failure. The higher strains in concrete beams also show that good bond between steel and concrete existed till the yielding of steel. The strains, before final failure may have been higher than the strains mentioned here. In Figure 3 strain is shown by “I”.

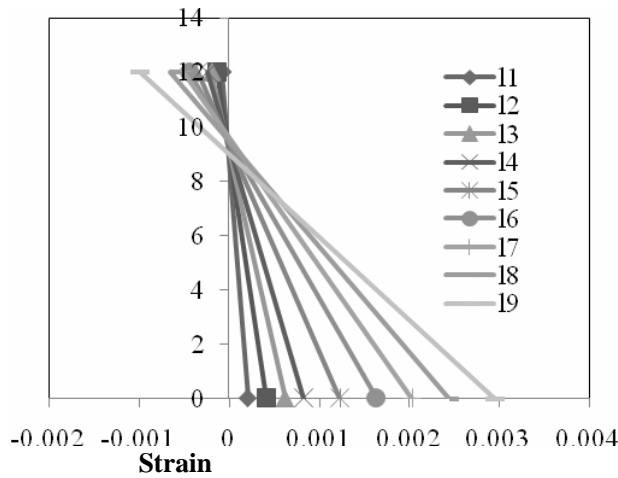


Figure 3. Depth vs. strain

7.2 Neutral Axis Depth

The study of the evolution of the neutral axis' depth from the start to the failure load and its relationship with the ductility is very important. One of the objectives of this study is to check if these variations follow the same tendencies already observed and accepted for RC beam with normal strength steel. The experimental values of the position of the neutral axis at the critical section, for a given beam and for each load level, were simply calculated by intersecting the line obtained by regression analysis from the experimental strains along the height of the section with the vertical axes. The value of the bending moment at the mid-span of each beam was statically calculated from the value of the total load applied to the beam at the considered load level. As mentioned earlier, the necessary ductility may be achieved through an idealization of the section so that the position of the neutral axis in failure, defined by the parameter NA (Neutral Axis Depth from top fiber), is limited to a certain maximum value. Such methodology is valid only for sections submitted to simple bending, such as the critical sections of the beams tested in this study. Therefore, the study of the evolution of the neutral axis' depth from the start to the failure load is very important. Also important is the value of the parameter NA in failure and its relationship with the ductility.

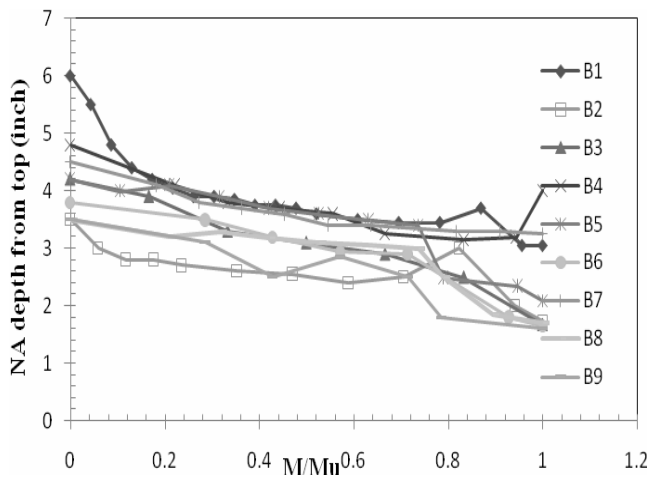


Figure 4. NA depth variation with moment ratio

The analysis of the graph (Figure 4) shows the existence of three distinct zones of the behavior of the neutral axis' depth throughout the tests, which correspond to a typical evolution of the curves. A first zone is identified by the rising of the neutral line as the load increases. The second zone corresponds to a stabilization of the neutral axis' depth as the beam suffers considerable deformations while the load increases slowly. Finally, the third behavior zone generally corresponds to an abrupt rise of the neutral axis up to the ultimate moment of the critical section. This behavior starts with the yielding of the longitudinal tensile reinforcement,

forcing the main crack to develop even further due to the sudden rise of the reinforcement strains up to the section failure.

7.3 Moment Curvature Curves

From Figures 5 to 8 it can be seen that the moment-curvature curves are almost linear before the peak moment is reached and there is a fairly long yield plateau at the post-peak stage. As the experiments were load control post yield plateau decrease of plastic moment could not be replicated. From moment-curvature relation information about flexural strength, flexural stiffness and more importantly flexural ductility can be extracted. Numerical moment-curvatures were calculated for the same cross section for all nine beams. These are compared with experimental moment curvature. Numerical results matched well with experimental result. Thus, numerical results may be used for further analysis as conducting experiment is always not feasible and costly too.

7.4 Role of Steel Ratio

It can be seen from the Figures 5 and 6 that both flexure strength and stiffness is increased with the increase of steel ratio. It can also be observed that low steel ratio gives more ductility than high steel ratio. In all cases the steel ratio is below the balanced steel ratio for that respective cross section. In ductility based design it is very important to keep the steel ratio low. It is true that high strength steel produces lower ductility than low strength steel. However, this loss of ductility can be compensated using lower steel ratio. High strength steel gives lower steel area for a particular moment and cross section which in turn ensure low steel ratio.

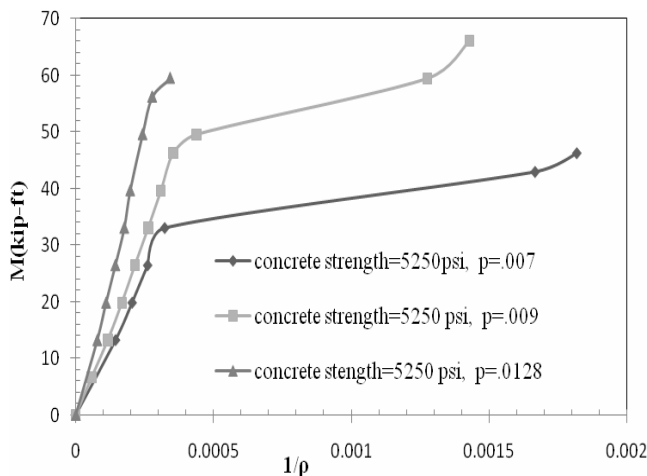


Figure 5. Moment curvature of different steel ratio for fixed concrete strength 5250psi (from experiment)

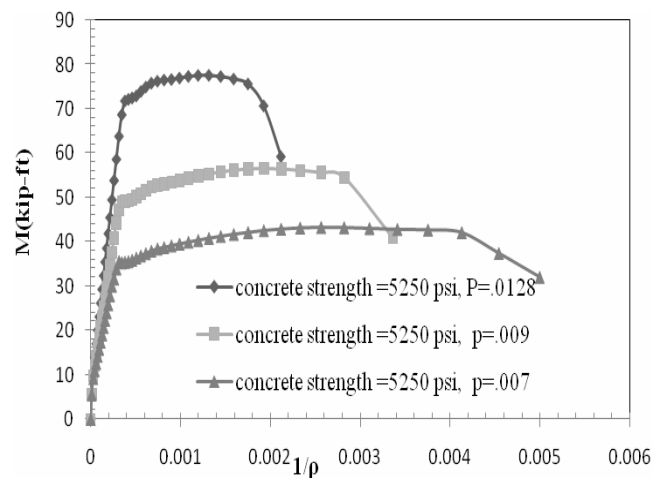


Figure 6. Moment curvature of different steel ratio for fixed concrete strength 5250psi (from experiment)

7.5 Role of Concrete Compressive Strength

Figures 7 and 8 shows the role of concrete compressive strength on moment-curvature relations. It can be seen from this figure that no significant effect of compressive strength of concrete on moment-curvature relationships were observed. This may be due to the fact that the strain controlled experiments could not be performed. Complete moment-curvature curve up to failure could not be traced. From ductility ratio some idea of concrete compressive strength is observed. It can be concluded that no significant change has been observed in both flexural strength and stiffness as compressive strength changes.

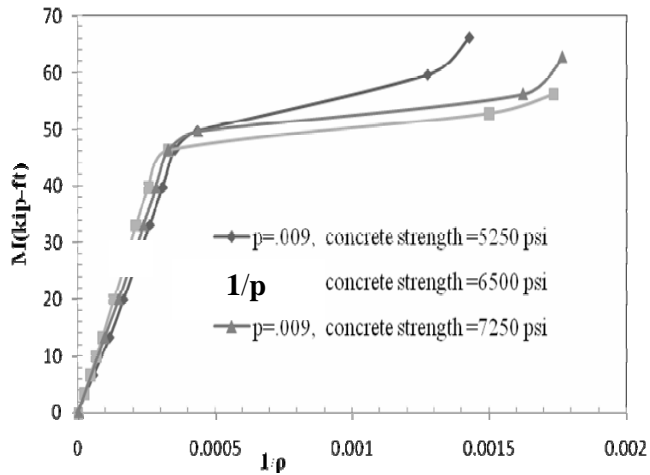


Figure 7. Moment curvature of different concrete strength for fixed steel ratio $p=0.009$ (from experiment)

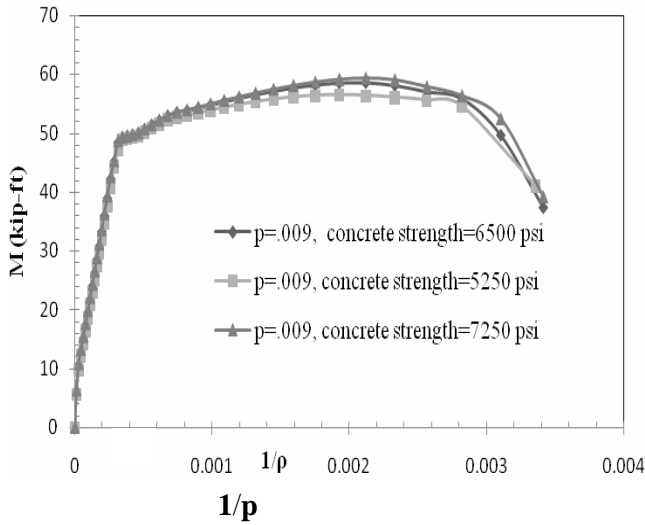


Figure 8. Moment curvature of different concrete strength for fixed steel ratio $p=0.009$ (from numerical analysis)

7.6 Ductility Ratio

Figure 9 shows the relationship of ductility ratio with concrete compressive strength for different steel ratio. It can be seen from the figure that no significant increase of ductility ratio is observed with concrete compressive strength. However, lower steel ratio gives higher ductility ratio. It is recommended from ductility based design that there must be a upper limit of steel ratio apart from p_{max} provided in different codes from ductility point of view.

From Figure 9 it is shown that minimum ductility level should be kept at ductility ratio 4. Beams with steel ratio 0.009 and 0.007 have ductility ratio more than 4. But the beam with steel ratio 0.0128 have ductility ratio less than 4. So low steel ratio beams give the required minimum level of flexural ductility.

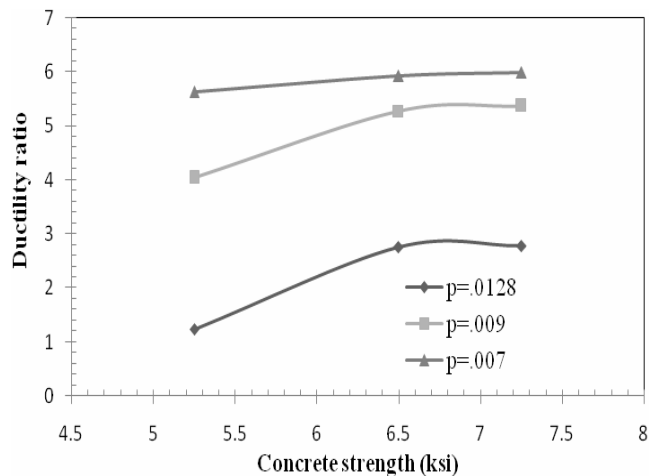


Figure 9. Change in concrete strength with ductility ratio

8 CONCLUSIONS

The use of a high strength steel would allow a higher flexural strength and stiffness to be achieved while maintaining the same minimum level of flexural ductility if steel ratio is properly selected. On the other hand, the use of a higher strength concrete would not allow a higher flexural strength and stiffness to be achieved while maintaining the same minimum level of flexural ductility; it only allows the use of a smaller steel area for a given flexural strength requirement to save the amount of steel needed and to avoid steel congestion. Proper bond between high strength concrete and high strength steel is observed even after post yield period. Strain curve and moment-curvature curve both validate the compatibility between the materials. From the tests, the evolution of the position of the neutral axis for high strength beams as the load increases, is identical to that reported for RC concrete beams with normal strength steel. The behavior of the formation and the growth of the cracks for normal-strength steel beams seem to be also valid for concrete beams using high-strength steel. There must be upper limit of steel ratio apart from ρ_{max} provided in codes of practice from ductility ratio point of view.

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