University of Cincinnati

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I, James M. Chamberlain Jr., hereby submit this original work as part of the requirements for the degree of Master of Science in Civil Engineering.

It is entitled:
Flexural Resistance Factors for Partially Prestressed Members Using ASTM A 1035 Reinforcing Steel

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Committee member: Richard Miller, PhD
Flexural Resistance Factors for Partially Prestressed Members Using
ASTM A1035 Reinforcing Steel

A thesis submitted to the
Graduate School
of the University of Cincinnati
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By
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B.S. University of Cincinnati, 2010

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ABSTRACT

The current standard reinforcing steel used in reinforced concrete beams in the United States is ASTM A615, Grade 60. However, several high-strength reinforcing bars are now available. These bars are believed to offer better performance in reinforced and prestressed concrete beams. The current specifications ACI 318 and AASHTO LRFD are designed for the use of A615, Grade 60 steel and do not account for the use of high strength steel in structures.

In reports published by the National Co-operative Highway Research Program Project 12-77 (Shahrooz et al.) and Mast et al. (2008) changes were recommended to AASHTO LRFD and ACI 318 for increasing the strain limit that defines tension-controlled members using high-strength steel. The codes state that a tension-controlled member is any member with strain in the extreme tensile steel greater than 0.005. This limit is independent of the steel type. NCHRP 12-77 and Mast et al. deemed that in order for members using high-strength steel to have the same behavior as members using A615, Grade 60 steel a larger strain limit for tension-control must be defined. Thus, future versions of ACI 318 and the AASHTO LRFD may contain strain limits that are dependent on steel type. This dependency will create situations in which tension control strain limits for members with mixed types of steel (most notably, partially prestressed members) will not be definable based on current codes. The reported research is aimed at establishing strain limits that define tension-controlled and compression-controlled behavior for partially prestressed members with high-strength reinforcing bars.

Response 2000 is analysis software that was used to determine the relationship between sections reinforced with A615 steel and those reinforced with A1035 steel. The relationship between curvature ductility and the strain in the extreme tensile steel was established for partially
prestressed members with A615 and A1035 reinforcing bars. For this purpose, 36 partially prestressed sections with A615 reinforcing bars were analyzed. For each case, the curvature ductility corresponding to the extreme tensile strain of 0.005 (the strain limit for tension-controlled members) was established. The same partially prestressed member was reanalyzed by using A1035 reinforcing bars. The extreme tensile strain corresponding to the previously calculated curvature ductility was determined. The same procedure was repeated but the extreme tensile strain was set equal to 0.002, which is the strain limit for compression-controlled members. In all the analyses, the code specified concrete compressive strain of 0.003 was used.

The analyses indicate that the extreme tensile strain has to be set equal to 0.0075 for partially prestressed members with A1035 reinforcing bars. This strain ensures the same level of ductility as that for partially prestressed members with A615 reinforcing bars. The extreme tensile strain defining compression-controlled behavior is 0.003 for partially prestressed members with A1035 instead of 0.002.
ACKNOWLEDGMENTS

I would first like to thank my advisor, Dr. Bahram Shahrooz, for introducing me to this project. Without his guidance and support this thesis would not have been possible. I would also like to thank Dr. Richard Miller and Julie Cromwell for providing their insight into this thesis.

Lastly, I am grateful for my family and friends, without their support this thesis would never have been completed.
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NOTATIONS

\( a \) depth of equivalent rectangular stress block
\( A_s \) area of steel reinforcement
\( A_{ps} \) area of prestressed-steel reinforcement
\( b \) width of beam
\( c \) distance from the extreme compression fiber to the neutral axis; diagonal distance to the nearest reinforcement in the section from the current depth
\( d \) distance from extreme compression fiber to centroid of tension reinforcement
\( d' \) distance from extreme compression fiber to centroid of compression reinforcement
\( E_c \) modulus of elasticity of concrete
\( E_s \) modulus of elasticity of steel
\( f'_c \) concrete compressive strength; unconfined concrete compressive strength
\( f_{pu} \) ultimate strength of prestressing strand
\( f_s \) stress in the tension reinforcement at nominal flexural resistance
\( f'_s \) stress in the compression reinforcement at nominal flexural resistance
\( f_y \) yield strength of reinforcing bars
\( f_u \) ultimate strength of reinforcing bars
\( h \) overall thickness or depth of the component
\( k \) function of a section’s reinforcement ratio, modular ratio and depth of steel
\( K_1 \) correction factor for source of aggregate
\( M_n \) nominal moment
\( n \) modular ratio
\( W_c \) concrete unit weight
\( y \) distance from the neutral axis
\( \beta_I \)  
ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone

\( \gamma \)  
Intensity modifier for the rectangular concrete compressive stress block

\( \varepsilon \)  
strain

\( \varepsilon_c \)  
strain of concrete in compression

\( \varepsilon_c' \)  
peak concrete strain

\( \varepsilon_s \)  
steel strain

\( \varepsilon_s' \)  
strain of steel reinforcement in compression

\( \varepsilon_t \)  
net tensile strain in extreme tension steel at nominal resistance

\( \varepsilon_x \)  
strain at neutral axis of section

\( \rho \)  
tensile reinforcement ratio

\( \rho' \)  
compressive reinforcement ratio

\( \mu_{\phi} \)  
curvature ductility

\( \Phi \)  
strength reduction factor

\( \phi \)  
curvature

\( \phi_{\text{design}} \)  
curvature for design loads

\( \phi_{\text{service}} \)  
curvature for service loads
CHAPTER 1: INTRODUCTION

The current standard reinforcing steel used in reinforced concrete beams in the United States is ASTM A615, Grade 60. However, several types of high-strength reinforcing bars are now available. These bars are believed to offer better performance in reinforced and prestressed concrete beams. The current specifications ACI 318 and AASHTO LRFD are designed for the use of A615, Grade 60 steel and do not account for the use of high-strength steel in structures.

In reports published by the National Cooperative Highway Research Program Project 12-77 (Shahrooz et al.) and Mast et al. (2008) changes were recommended to AASHTO LRFD and ACI 318 for increasing the strain limit that defines tension-controlled members using high strength steel. The codes state that a tension-controlled member is any member with strain in the extreme tensile steel greater than 0.005. This limit is independent of the steel type. NCHRP 12-77 and Mast et al. deemed that in order for members using high-strength steel to have the same behavior as members using A615, Grade 60 steel a larger strain limit for tension-control must be defined. Thus, future versions of ACI 318 and the AASHTO LRFD may contain strain limits that are dependent on steel type. This dependency will create situations in which tension control strain limits for members with mixed types of steel (most notably, partially prestressed members) will not be definable based on current codes.

The tension-control strain limit is important because it is used in determining the strength reduction factor for flexure, Φ. The current code states that a Φ factor of 0.65 is used for compression-controlled members and 0.90 for tension-controlled members, while values of Φ that lie in the transition zone are interpolated between 0.65 and 0.90. If the minimum strain level for tension-control is raised for members containing high-strength steel, a new method for
defining Φ must be developed. Otherwise, a member having an extreme tensile strain of 0.005 would be considered tension-controlled and would be given a value of Φ of 0.90, when it is actually in the transition zone and should be assigned a lower value of Φ.

CHAPTER 2: PROBLEM STATEMENT

A number of recent studies have made recommendations for new tension-control strain limits using high strength steel. However, these studies do not address the use of high strength steel in partially prestressed members. The current code uses the same tension-control limit for non-prestressed and prestressed members. Considering that a change has been proposed for non-prestressed members using high strength steel, a change also needs to be proposed for partially prestressed members using high strength, non-prestressed steel. Moreover, if the minimum tension-control limit is raised, a new method of defining strength reduction factor for flexure must be developed.

CHAPTER 3: BACKGROUND

ACI 318 defines the flexural strength reduction factor, Φ, based on the strain in the extreme tensile steel at the nominal moment capacity. Research by Mast et al. (2008) and Ward (2008) concluded that the strain limits for members with A615 steel were not applicable for members with high-strength steel. Their research included similar research techniques. Both Mast et al. and Ward created a reinforced concrete section using A615 reinforcing steel. They designed the section so that when the nominal moment capacity was reached the strain in the extreme tensile steel was 0.005. The ductility of the section was then found, where ductility was defined as the ratio of curvature, deflection, or strain at the nominal flexural capacity (Mn) to the
same response at service moment. The analysis was repeated with the A615 reinforcing bars being replaced by high-strength reinforcement. The amount of high-strength steel in the section was varied until the section had the same ductility as the section with the A615 steel. Once the matching ductility was achieved the strain in the steel was recorded for the section and \( M_n \). Mast et al. confined their research to only one member with A1035 steel. For that member they examined deflection, curvature, and strain ductility. Ward used several sections with different steel types but only examined curvature ductility. Mast et al. (2008) and Ward (2008) recommended that the strain limit for compression-controlled members be raised to 0.004 for members using high strength steel. Both of their recommended strain limits for tension-controlled members were increased. However, Mast et al. recommended a limit of 0.009 whereas Ward recommended a limit of 0.008. The strain limits were adjusted to allow for an elastic-perfectly plastic model to be used for high-strength steel.

The limits of Mast et al. and Ward’s studies are that only reinforced sections were considered. Hence, there remains a need for further analysis in partially prestressed sections that use nonprestressed high-strength reinforcement as tensile steel.

CHAPTER 4: PROCESS

4.1 Overview

Similar to the work of Mast (2008) and Ward (2008), a formulation was generated to compute new strain limits for partially prestressed members using A1035 reinforcing steel. This formulation is based on achieving the same level of curvature ductility for partially prestressed members with A615 or A1035 reinforcing bars.
4.2 MATLAB Code

Ward developed a spreadsheet to calculate the curvature ductility of reinforced concrete members while varying several member parameters. The process Ward used to create this spreadsheet can be found in Appendix C.

Using Ward’s EXCEL program as a template, a MATLAB program (see Appendix E) was created to more effectively analyze different members. The program analyzed a reinforced concrete beam with A615 steel and determined the member’s curvature ductility. The same beam was then analyzed using varying amounts of A1035 steel until the curvature ductility of the first beam was matched. The corresponding strain in the extreme tensile steel was then recorded.

One assumption used in the formulation of the MATLAB code was that the member had cracked at service and design loadings. This assumption allowed for the code to be non-section sensitive as long as the compressive stress block was rectangular. However, this assumption cannot be applied to partially prestressed members because they are designed not to crack under service loads. If the member has not cracked, the calculations become section specific. In lieu of altering the MATLAB code for each partially prestressed section being examined, it was determined that the use of the analysis software Response 2000 (Bentz & Collins) would be more efficient.

4.3 Response 2000

4.3.1 Formulation

Proposed strain limits for partially prestressed members using A1035 reinforcing steel were developed as follows.
1. For a given section, value of $A_{ps}$ and $A_s$, use Response 2000 to analyze the section and generate the moment – curvature and moment – neutral axis strain plots. Generic relationships are shown in Figure 4.3.1.

![](image)

(a) Moment-curvature plot  
(b) Moment vs. neutral axis strain plot

**Figure 4.3.1.1 Analysis Results from Response 2000**

2. The strain shown in Fig. 4.3.1.1(b) is that at the neutral axis. Knowing the curvature ($\phi$) and strain at the neutral axis ($\varepsilon_x$), EQ-4.1 can be used to compute strain at any other elevation ($y$), in particular the extreme tensile strain ($\varepsilon_t$) and the extreme fiber concrete compressive strain. This equation is based on the assumption that plane sections before bending remain plane after bending.

$$\varepsilon = \varepsilon_x - \phi \cdot y$$

(EQ-4.1)

3. Use interpolation to determine the design curvature corresponding to extreme fiber concrete compressive strain equal to 0.003; the service curvature corresponding to tensile strain at 60% of the yield strength (i.e., $0.6f_y$); and the extreme tensile strain in the mild reinforcement corresponding to extreme concrete compressive strain equal to 0.003. Knowing the design and service curvatures, compute an implied curvature ductility as $\mu_\phi = \phi_{\text{design}} / \phi_{\text{curvature}}$. 

---

<table>
<thead>
<tr>
<th>Moment (ft-kips)</th>
<th>Curvature (rad/10^6 in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment (ft-kips)</td>
<td>Strain ($\mu\varepsilon$)</td>
</tr>
</tbody>
</table>
4. Repeat steps (1) to (3) while varying the $A_{ps}$ and $A_s$ to establish a relationship between the curvature ductility and extreme tensile strain ($\varepsilon_t$). Such a relationship is shown in Figure 4.3.2.2. The analyses were conducted in such a way that $\mu_\phi - \varepsilon_t$ relationship includes the current compression and tension-control strain limits, i.e., 0.002 and 0.005.

![Curvature Ductility vs. Steel Strain Graph]

**Figure 4.3.2.2 Curvature Ductility vs. Steel Strain at Design**

5. Using interpolation determine the curvature ductility that corresponds to the compression-control strain limit, 0.002, and the tension control strain limit, 0.005.

6. Repeat steps (1) to (4) using A1035 in place of A615 steel, ensuring that the relationship found in step (4) includes the curvature ductilities that correspond to those for compression-control and tension-control strain limits of A615 mild reinforcement.

7. Use interpolation to determine the strain in the A1035 mild reinforcement that corresponds to the curvature ductility at which the strain in the A615 reinforcement is 0.002 (i.e., compression-control strain limit) and the curvature ductility corresponding to strain of 0.005 in the A615 reinforcement (i.e., tension-control strain limit).
4.3.2 Response 2000 Material Model

The A615 and A1035 reinforcement was to behave elastic-perfectly plastic, which is consistent with the basis of current definitions of tension-controlled and compression-controlled strain limits (Mast 1992). For this purpose, the ultimate strength was selected to match the yield strength, refer to Figure 4.2.1. As shown in Figure 4.2.2, the resulting stress-strain curves are elastic-perfectly plastic.

The initial strain applied to the prestressing was set to 0.0055. Assuming that the strand is tensioned to an initial pull of 0.75f_{pu} the initial strain accounts for 21% losses in prestressing.
Figure 4.2.1 Steel Material Properties in Response 2000 for A615 and A1035

Rebar

\[ f_u = 60 \text{ ksi} \]

\[ f_y = 60 \]

\[ \varepsilon_s = 100.0 \text{ ms} \]

Rebar

\[ f_u = 100 \text{ ksi} \]

\[ f_y = 100 \]

\[ \varepsilon_s = 100.0 \text{ ms} \]

Figure 4.2.2 Elastic-Plastic Model for A615 and A1035 Steel
For the concrete model, the peak strain (corresponding to the compressive strength) was set equal to 0.003. Consistent with current design practice, the concrete tensile strength was neglected and tension-stiffening factor was set equal to zero. A representative set of parameters for 10,000-psi concrete is shown in Figure 4.2.3 with the resulting concrete stress-strain relationship illustrated in Figure 4.2.4.

![Materials Page](image)

**Figure 4.2.3 Example of Concrete Material Properties in Response 2000**

![Concrete Stress-Strain Model](image)

**Figure 4.2.4 Example Stress-Strain Model for Concrete**
4.3.3 Model Verification

The aforementioned procedure was validated by comparing the results for a control section against those obtained from the MATLAB code discussed in Section 4.2. The selected section was a 24 inch x 24 inch reinforced concrete beam, with $f'_c$ of 5, 10, and 15 ksi and varying values of $A_s$ similar to the procedure in section 4.3.1. As seen in Figure 4.3.3, the results from the two procedures are similar. The difference of less than 5% is deemed to be negligible and can be attributed to differences in Response 2000 and the MATLAB code’s analysis techniques.

![Figure 4.3.5 Equivalent Strains: Matlab vs. Response 2000](image)

4.3.4 Section Analysis in Response 2000

Response 2000 was used to analyze 36 cross sections shown in Table 4.3.1. The strands had the typical strand grid pattern of 2”x2”. The cross sections of the selected members are shown in Appendix D. Partially prestressed members had nonprestressed reinforcement (A615 or A1035) equal to 20% of the total area of prestressing reinforcement ($A_{ps}$). The tensile
reinforcement was placed at the elevations corresponding to the strands that had been replaced by nonprestressed reinforcement; the compression reinforcement was at 2” from the top of the section. Compression steel is added to the section to prevent tensile cracking in the top of the section at initial release.

Table 4.3.1 Range of Prestressing Strands Used in Each Section

<table>
<thead>
<tr>
<th>Section</th>
<th>A615 Steel Range of Prestressing Strands</th>
<th>A1035 Steel Range of Prestressing Strands</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5 ksi</td>
<td>10 ksi</td>
</tr>
<tr>
<td>AASHTO Type I</td>
<td>8 - 14*</td>
<td>14 - 33*</td>
</tr>
<tr>
<td>AASHTO Type II</td>
<td>12 - 20*</td>
<td>21 - 40*</td>
</tr>
<tr>
<td>AASHTO Type IV</td>
<td>28 - 49*</td>
<td>50 - 102*</td>
</tr>
<tr>
<td>AASHTO Type V</td>
<td>45 - 75*</td>
<td>84 - 98</td>
</tr>
<tr>
<td>AASHTO Type VI</td>
<td>34 - 51</td>
<td>64 - 114</td>
</tr>
<tr>
<td>PCI 12RB20</td>
<td>7 - 14*</td>
<td>12 - 29*</td>
</tr>
<tr>
<td>PCI 12RB36</td>
<td>6 - 27*</td>
<td>20 - 55*</td>
</tr>
<tr>
<td>BT-54</td>
<td>22 - 32</td>
<td>44 - 71</td>
</tr>
<tr>
<td>BT-63</td>
<td>22 - 41</td>
<td>46 - 76</td>
</tr>
<tr>
<td>BT-72</td>
<td>24 - 37</td>
<td>48 - 82</td>
</tr>
</tbody>
</table>

N/A means that the section was unable to reach the compression-control limit. (*) Denotes S.5 strand.

Sections with large compression flanges such as PCI single tee and double tee beams are not shown in this analysis because it was determined that they would always be above the tension-control strain limit. Several of these sections were checked; for all cases, none of the members could be brought below the tension-control strain limit, refer to Appendix F. In addition to these sections, it was determined that members with a composite deck were also unable to reach the tension control limit; hence, they were not included in this research. It is understood that most of the sections analyzed will always have a composite deck. The focus of the presented research was to modify the tension-control and compression-control strain limits.
for partially prestressed members with high-strength reinforcing bars. Hence, only sections that could reach each limit were examined.

CHAPTER 5: RESULTS

A representative relationship between curvature ductility and extreme tensile strain is plotted in Figure 5.1. This graph is a partially prestressed AASHTO Type I girder using 10-ksi concrete. To achieve the same level of ductility corresponding to a tension-controlled partially prestressed member with A615 reinforcing bars, the extreme tensile strain has to be changed from 0.005 to 0.0075 if the nonprestressed reinforcement is A1035. When A1035 reinforcement is used, the strain defining compression-controlled has to be 0.00315 (instead of 0.002 for A615 reinforcement).

![Figure 5.1 Example AASHTO Type I](image)

The results for all 36 cases can be seen in Figure 5.2. The trend lines show the average values of strain limits for all 36 members. These results show that even though the sections
change there is little to no variation in the equivalent strains. Figures 5.3 and 5.4 show an enlarged view of the results, and Table 5.1 shows the statistical information of the results. Table 5.1 indicates that the variations in equivalent strain are very small and are no cause for concern.

**Figure 5.2** Equivalent strains for tension-controlled and compression-controlled members reinforced with ASTM A1035.

**Figure 5.3** Equivalent strains for tension-controlled members reinforced with ASTM A1035
Equivalent strains for compression-controlled members reinforced with ASTM A1035

Table 5.1 Strain Limit Statistical Information

<table>
<thead>
<tr>
<th>$f'_c$</th>
<th>Tension-Control</th>
<th>Compression-Control</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
<td>Std. Dev.</td>
</tr>
<tr>
<td>5ksi</td>
<td>0.007671</td>
<td>5.2706E-05</td>
</tr>
<tr>
<td>10ksi</td>
<td>0.007562</td>
<td>5.21957E-05</td>
</tr>
<tr>
<td>15ksi</td>
<td>0.007456</td>
<td>8.50059E-05</td>
</tr>
</tbody>
</table>

CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

The reported research investigated the compression-control and tension-control strain limits of high-strength steel, Gr. 100 A1035, used as non-prestressed steel in partial-prestressed reinforced concrete. Currently the code uses the same strain limits for mild and high strength steel, which could change in the future. Previous work done by Mast et al. (2008) and Ward (2008) suggest that the strain limits for compression and tension-control have to be increased for
members using high-strength reinforcement. If the code chooses to adopt these recommendations, the strain limits will become dependent on steel strength. This dependency will create a problem for members using different grades of reinforcement, in particular for partially-prestressed girders. Therefore, this research was performed in order to determine strain limits of section with multiple yield strength reinforcing.

6.2 Conclusions

The results suggest that the tensile strains defining tension-control and compression-control for partially prestressed members with A1035 reinforcement have to be adjusted. The following strain limits are recommended for partially prestressed members.

Tension-Controlled: \( \varepsilon_t \geq 0.0075 \)

Compression-Controlled: \( \varepsilon_t \leq 0.003 \)

Figure 6.2.1 Variation in \( \Phi \) Based on Extreme Tensile Steel Strain

6.3 Recommendations for Future Research

The research in the paper was limited due to the lack of experimental data. It is recommended that the analytically generated strain limits be experimentally verified.
Furthermore, curvature ductility was used as the metric to establish revised strain limits. Additional studies are necessary to also utilize deflection and strain ductility.

Finally, other types of high-strength reinforcement such as A955 and A706 need to be examined.
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APPENDIX A: LIST OF SECTIONS AND THEIR CORRESPONDING STRAIN LIMITS

Table A-1 Equivalent strains for tension-controlled and compression-controlled members.  
\( f'_c = 5\text{ksi} \) and \( f_y = 100\text{ksi} \)

<table>
<thead>
<tr>
<th>Member</th>
<th>( f'_c ) (ksi)</th>
<th>Strain</th>
<th>Member</th>
<th>( f'_c ) (ksi)</th>
<th>Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO Type I</td>
<td>5</td>
<td>0.00760</td>
<td>AASHTO Type I</td>
<td>5</td>
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Table A-2 Equivalent strains for tension-controlled and compression-controlled members.

\( f'_c = 10\text{ksi} \) and \( f_y = 100\text{ksi} \)

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<tr>
<th>Member</th>
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<th>Compression Control</th>
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<td>( f'_c ) (ksi)</td>
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Table A-3 Equivalent strains for tension-controlled and compression-controlled members.

\( f'_c = 15\)ksi and \( f_y = 100\)ksi

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<th>Member</th>
<th>( f'_c ) (ksi)</th>
<th>Strain</th>
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APPENDIX B: EXCEL SPREAD SHEET FOR RESPONSE DATA REDUCTION

The data from the Response 2000 Analysis were analyzed using an Excel workbook created by the author of this document. The Excel file was used to determine the curvature ductility of a section based on the analysis output from Response 2000.

The file opens to an “Overview” tab. On this tab, the user inputs the beam type, concrete compressive strength, size of prestressing strand, depth from the neutral axis to desired strain, i.e., depth of extreme tensile steel, and the distance from the neutral axis to the top of concrete. The user must also input the number of strands for the particular run that is being investigated. In Figure B-1, it can be seen that for Beam 1 the number of strands in the section is 10. The spreadsheet uses the number of strands to determine what 20% of the $A_{ps}$ is, and outputs that number in the cell marked $A_s$. The user then modifies the area of tensile and compressive non-prestressing steel in the section to match the value for 20% $A_{ps}$ and runs the analysis. From the Response 2000 results the user will copy 2 tables of data.

An AASHTO Type I section using 15ksi concrete strength is used as an example of how the spreadsheet works. First the user must generate a section in Response 2000. Once the section is created, the user opens the Excel file and input the section properties. The Excel “Overview” tab is shown in Figure B-1. In this sheet, the user inputs the beam type in cell C3, the concrete strength in cell C4, the depth from the neutral axis to the extreme tensile steel in cell I3, the distance from the neutral axis to the top of concrete in cell I4, and for this example the number of strands in the section is input in cell B9 which corresponds to Beam 1. Once the number of strands in the section is input, the Excel sheet calculates the 20% of prestressing that is to be used as mild steel. The user then inputs the strands and mild top and bottom steel into the section.
Figure B-1 Excel Workbook Overview Tab

Once the section is completed in Response, the analysis is run. From the Response analysis results two sets of data are selected. The first is the M-phi, and the second is the M-ex plot. The user right clicks the plot in Response and select the “Copy Chart Data” option. Once the data are copied, they are pasted into the Excel tab titled “I-1”, shown in Figure B-2. In the I-1 tab, there are two gray boxes that are labeled for pasting each set of data. The user clicks on the grey box corresponding to the data copied from Response and then select the Text Import.
Wizard to import the text properly. In the Text Import Wizard menu, the user selects comma and space as the delineators.

![Excel Input Tab](image)

**Figure B-2 Example of Excel Input Tab**

After the data from the two plots are input into the Excel sheet, it is reduced and the results are shown on the tab labeled “O-1”, shown in Figure B-3. In this tab, the user can see the values of service and design curvature, as well as the value for curvature ductility and the steel strain that corresponds to the design curvature. These values are gathered for all of the beams, one through 20, and the results are shown in the “Results” tab, Figure B-4, at the end of the spreadsheet. The user must use sort function to numerically arrange the values of steel strain in
order to have the interpolation function on the “Results” tab work correctly. From the graph the user can pull the equivalent 100-ksi steel strains.

<table>
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<tr>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
<th>H</th>
<th>I</th>
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<td>Moment (kip-ft)</td>
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Figure B-3 Example of Excel Output Tab
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<th>Beam</th>
<th>Curvature Ductility at</th>
<th>Strain in 100ksi steel</th>
<th>at matching curvature</th>
<th>ductility of 60ksi</th>
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**Figure B-4 Excel Workbook Results Tab**
APPENDIX C: TENSION AND COMPRESSION-CONTROLLED STRAIN LIMITS FOR ASTM A1035

C.1 Introduction

As shown in Figure B1, the strength reduction factor for flexural members varies depending on the expected behavior. The strain in the extreme tension reinforcement \( \varepsilon_t \) controls whether the behavior will be tension-controlled, compression-controlled, or in transition zone.

![Figure C1 Strength Reduction Factors for Flexural Members](image)

The current steel strain limits of 0.005 defining the lower bound of tension-controlled behavior and 0.002 or less defining compression-controlled behavior are based on having an adequate change in steel strain from service load to nominal strength. The current strain limit of 0.005 defining tension-controlled behavior is based on having an adequate change in steel strain from service load to nominal strength. This concept is shown in Figure B2 for Grade 60 reinforcement with and without a well-defined yield point. The additional strain beyond service load provides warning at high load levels, which is the basic characteristic of tension-controlled
flexural members.

Nonetheless, the strain limits have been calibrated based on the expected performance of flexural members reinforced with Grade 60 longitudinal bars. Considering that A1035 bars could be subjected to larger service level strains and have different stress-strain relationships, the strain limits defining tension-controlled and compression-controlled behaviors need to be reevaluated.

C.2 Formulation

Proposed strain limits for A1035 reinforcing steel were developed as follows.

1. For a given value of $f'_c$, $\rho$, $\rho'$, and $d'/d$, calculate the tensile steel strain ($\varepsilon_s$) required to maintain the equilibrium of forces shown in C3.

Figure C2 Illustration of Change in Strain (Adapted based on Mast 1992)
Consistent with the basis of current definitions of tension-controlled and compression-controlled behaviors (Mast 1992), an elastic-perfectly plastic model is used for ASTM A615 bars, i.e.,

\[
\begin{align*}
\text{if } \varepsilon_s \leq \varepsilon_y & \quad f_s = E_s \varepsilon_s \left( \varepsilon_s' \leq \varepsilon_y \right) = E_s \varepsilon_s' \\
\text{if } \varepsilon_s > \varepsilon_y & \quad f_s = f_y \left( \varepsilon_s' > \varepsilon_y \right) = f_y
\end{align*}
\]

Using basic principles, the following equations are obtained to maintain equilibrium:

If the top bars are compressive:

\[
\gamma f'_c \beta_1 \frac{\varepsilon_c}{\varepsilon_c + \varepsilon_s} = \rho f_s - \left( \frac{d'}{d} \right) \rho' \left( f'_s - \gamma f'_c \right) \tag{C-1}
\]

If the top bars are tensile:

\[
\gamma f'_c \beta_1 \frac{\varepsilon_c}{\varepsilon_c + \varepsilon_s} = \rho f_s + \left( \frac{d'}{d} \right) \rho' f'_s \tag{C-2}
\]

Note that \( c = \frac{\varepsilon_c}{\varepsilon_c + \varepsilon_s} d \) and \( \varepsilon_s' = \varepsilon_c \left( 1 - \frac{d'}{d} \right) - \left( \frac{d'}{d} \right) \varepsilon_s \)
According to NCHRP 12-64 (Rizkalla et al., 2007), use the following concrete parameters:

\[
\beta_1 = \begin{cases} 
0.85 & \text{for } f'_c \leq 4 \text{ksi} \\
0.85 - 0.05(f'_c - 4) \geq 0.65 & \text{for } f'_c > 4 \text{ksi}
\end{cases}
\]

\(f'_c \text{ in ksi}\)

\[
\gamma = \begin{cases} 
0.85 & \text{for } f'_c \leq 10 \text{ksi} \\
0.85 - 0.02(f'_c - 10) \geq 0.75 & \text{for } f'_c > 10 \text{ksi}
\end{cases}
\]

\(\epsilon_c = 0.003\)

Obtain the value of \(\epsilon_s\) required to satisfy Eq. (C-1) or (C-2).

2. Repeat Step (1) for \(f'_c = 4\) to 15 ksi (in 1 ksi increments), \(\rho\) (0.1\% to 6\% in 0.06\% increments), \(\rho'\) (taken as 0, 0.5\(\rho\), and \(\rho\)), and \(d'/d = 0\) or 0.1.

3. For each of the cases discussed in Step (2), obtain an implied curvature ductility with

\[
\mu = \frac{\epsilon_c}{d - kd} = \frac{\epsilon_c d}{\epsilon_c + \epsilon_s} = \frac{(\epsilon_c + \epsilon_s)(1 - k)}{f_s/E_s}
\]

which \(\epsilon_c = \) strain computed in Step (2); and \(k = \sqrt{(\rho + \rho')^2 n^2 + 2(\rho + \rho') d'/d} n - (\rho + \rho')n\)

where \(f_s\) is service load stress, taken as 0.60\(f_y\) (to be consistent with the original development of strain limits for tension-controlled and compression-controlled responses), \(n = E_s/E_c\), and \(E_c = 310,000 K_1 \left(W_c^{-0.5} f'_c\right)^{0.33}\) with \(K_1 = 1, W_c = 0.15 kcf' = ksi units\) according to NCHRP 12-64.
4. Establish the implied values of curvature ductility at $\varepsilon_s = 0.005$ and $0.002$.

5. Repeat Steps (1) to (4) for ASTM A1035 bars with the following modifications:

Use Mast equation (Mast, 2006) to model steel stress-strain relationship, i.e.,

\[
if \ \varepsilon_s \leq 0.00241 \ f_s = E_s \varepsilon_s \ (\varepsilon_s \leq 0.00241 \ f_s = E_s \varepsilon_s)
\]

\[
if \ \varepsilon_s > 0.00241 \ f_s = 170 - \frac{0.43}{\varepsilon_s + 0.00188} \left(\varepsilon_s > 0.00241 \ f_s = 170 - \frac{0.43}{\varepsilon_s + 0.00188}\right)
\]

In addition, assume $f_y = 100$ ksi; hence, $f_s = 60$ ksi (instead of 36 ksi used in Step 3).

As evident from Figure C5, Mast equation provides a lower-bound estimate of the measured stress-strain diagrams of all A1035 bars tested as part of this part.

6. Using the curvature ductility-strain relationships obtained in Step 5 and Step 6, determine the steel strains corresponding to the implied curvature ductility calculated in Step 4.
The relationship between the strain levels for A615 and A1035 reinforcing bars is illustrated in Figure C6 for one of the 144 cases considered. For this example, a singly reinforced member having $f'_c = 4$ ksi, the strain in the A1035 bars needs to be 0.00793 in order to achieve the same implied ductility of the same tension-controlled member reinforced with A615 bars.
Figure C6 Example for $f'_{c} = 4$ ksi, $\rho' = 0$, $d'/d = 0$, target $\varepsilon_t = 0.005$ in ASTM A615

The complete set of results is shown in Figure C7. Each data point in this figure was established based on the methodology illustrated in Figure C6. As expected, the addition of compression bars (i.e., $\rho' > 0$) increases the strain in the tension reinforcement, which improves the ductility. As the concrete compressive strength increases, the tension reinforcement strain drops, which is an indication of reduced ductility.
The original research (Mast, 1992) used as the basis of defining strains for tension and compression controlled members did not account for the benefits of compression reinforcement. A similar conservative approach was followed herein. Based on the results shown in Figure C7, the following strains are recommended to define tension-controlled and compression-controlled members that use ASTM A1035 with the service load stresses limited to 60 ksi or less.

**C.3 Recommendations**

Tension-Controlled: \( \varepsilon_t \geq 0.008 \)

Compression-Controlled: \( \varepsilon_t \leq 0.004 \)
APPENDIX D: STRAND PATTERNS FOR SECTIONS USED IN ANALYSIS

Three types of sections were analyzed for this project. Two of these sections were typical AASHTO sections and the third was a rectangular section. The strand patterns used in analysis for the AASHTO I-Beams and the AASHTO-PCI Bulb-Tees can be seen in Figure D-1 and Figure D-2 respectively. The strand pattern used for the 3 rectangular sections was that of a 2” x 2” grid with a minimum of 1.5” clearance on all side.
Figure D-1 PCI Typical Strand Patterns for AASHTO I-Beams (PCI Bridge Design Manual)
Figure D-2 PCI Typical Strand Patterns for AASHTO-PCI Bulb-Tees (PCI Bridge Design Manual)
## RECTANGULAR BEAMS

### PCI Rectangular Beam Section Properties (PCI Bridge Design Manual)

#### Rectangular Beam Section Properties

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<th>b (in)</th>
<th>h (in)</th>
<th>A (in.²)</th>
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<th>Y_y (in)</th>
<th>S_y (in.³)</th>
<th>wt (plf)</th>
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1. Check local area for availability of other sizes.
2. Safe loads shown include 50% superimposed dead load and 50% live load.
3. 800 psi top tension has been allowed, therefore, additional top reinforcement is required.
4. Low-relaxation strand.

#### Table of safe superimposed service load (plf) and cambers (in.)

| Designation | Span, ft | y_y(end) in | y_y(center) in | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 40 | 42 | 44 | 46 | 48 | 50 | 52 |
|-------------|----------|-------------|----------------|-----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| 12RB16      | 58-S     | 3.00        | 3.00           | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 |
|             | 58-S     | 3.00        | 3.00           | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 |
| 12RB20      | 88-S     | 3.00        | 3.00           | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 |
| 12RB24      | 108-S    | 3.60        | 3.60           | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 |
| 12RB28      | 128-S    | 4.00        | 4.00           | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 |
| 12RB32      | 138-S    | 4.77        | 4.77           | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 |
| 12RB36      | 158-S    | 5.07        | 5.07           | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 |
| 16RB24      | 138-S    | 3.54        | 3.54           | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 |
| 16RB28      | 148-S    | 3.71        | 3.71           | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 |
| 16RB32      | 188-S    | 4.67        | 4.67           | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 |
| 16RB36      | 208-S    | 5.40        | 5.40           | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 |
| 16RB40      | 228-S    | 6.00        | 6.00           | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 | 95 |

Key:
- 3553 – Safe superimposed service load, plf.
- 0.4 – Estimated camber at erection, in.
- 0.2 – Estimated long-time camber, in.

Figure D-3 PCI Rectangular Beam Section Properties (PCI Bridge Design Manual)
APPENDIX E: MATLAB CODE

E.1 Matlab code

clear all
clc
cnt=0;

%This program uses an input file to run a series of calculations in order
%to determine the strain limit for tension control using high strength
%reinforcing steel

clearoutput %calls on script that deletes the existing output values

format long e

c=[0:0.0005:.5]; % c is a range of values for the steel strain
rho=[0.001:0.0006:.2];

x=length(c);
z=length(rho);
d=1;
dp=0.1;
for i=1:x
    fs60(i)=min(29000*c(i),60);
    fs100(i)=min(29000*c(i),100);
\[ cp(i) = 0.003(1 - dp/d) - (dp/d) \cdot c(i); \]
\[ cprime(i) = \text{abs}(cp(i)); \]
\[ \text{if dp==0} \]
\[ \quad cprime(i) = 0; \]
\[ \text{end} \]
\[ fs60prime(i) = \text{min}(29000 \cdot cprime(i), 60); \]
\[ fs100prime(i) = \text{min}(29000 \cdot cprime(i), 100); \]
\[ \text{end} \]

\[
\text{fid=fopen('concrete_inputs.txt');}\\
\text{for } i=1:72\\
\quad h=\text{textscan(fid,'%.3f %.3f %.3f %.3f %.3f %.3f',1);}\\
\quad fcprime=h\{1\};\\
\quad beta1=h\{2\};\\
\quad gamma=h\{3\};\\
\quad rhop=h\{4\};\\
\quad dprime=h\{5\};\\
\quad straincontrol=h\{6\}; %\text{value for tension/compression control}\\
\]

\[
\text{for } i=1:x\\
\quad \text{for } k=1:z
\]
rhoprime=rho.*rhop;
K1=1;
Wc=0.15;
Ec=310000*K1*Wc^2.5*(fcprime)^(1/3);
n=29000/Ec;
a(i)=beta1*(0.003/(0.003+c(i)))*d;
k1(k)=sqrt((rho(k)+rhoprime(k))^2*(n)^2+2*(rho(k)+rhoprime(k)*dprime/d)*n)-
         (rho(k)+rhoprime(k)))*n;
if a(i)>0.1*d
         C60_2(i,k)=gamma*fcprime*beta1*(0.003/(0.003+c(i)))-
rho(k)*fs60(i)+rhoprime(k)*(dprime/d)*(fs60prime(i)-gamma*fcprime);
         C100_2(i,k)=gamma*fcprime*beta1*(0.003/(0.003+c(i)))-
rho(k)*fs100(i)+rhoprime(k)*(dprime/d)*(fs100prime(i)-gamma*fcprime);
else
         if cp(i)>=0
         C60_2(i,k)=gamma*fcprime*beta1*(0.003/(0.003+c(i)))-
rho(k)*fs60(i)-rhoprime(k)*(dprime/d)*fs60prime(i);
         C100_2(i,k)=gamma*fcprime*beta1*(0.003/(0.003+c(i)))-
rho(k)*fs100(i)-rhoprime(k)*(dprime/d)*fs100prime(i);
else
         C60_2(i,k)=gamma*fcprime*beta1*(0.003/(0.003+c(i)))-
rho(k)*fs60(i)-rhoprime(k)*(dprime/d)*fs60prime(i);
         C100_2(i,k)=gamma*fcprime*beta1*(0.003/(0.003+c(i)))-
rho(k)*fs100(i)-rhoprime(k)*(dprime/d)*fs100prime(i);
end
end
end
end
for k=1:z
    es60_2(k)=interp1(C60_2(:,k),c,0,'linear');
    curvduct60_2(k)=((0.003+es60_2(k))*(1-k1(k)))/(36/29000);
    es100_2(k)=interp1(C100_2(:,k),c,0,'linear');
    curvduct100_2(k)=((0.003+es100_2(k))*(1-k1(k)))/(60/29000);
end

% Interpolation function to find the curvature dutility corresponding to
% 0.005 strain for the 60ksi steel. That curvature ductility is then used to
% find which strain it correspondse to for the 100ksi steel

cduct60_1=interp1(es60_2,curvduct60_2,straincontrol,'linear');
estrain100_1=interp1(curvduct100_2,es100_2,cduct60_1,'linear');

%------------------------------------------------------------------------
cnt=cnt+1;
if cnt <= 12
    fid1=fopen('output1.txt','a');  %output1 contains the strain values for 100ksi steel
    fprintf(fid1,'%.5f
',estrain100_1);
end
if cnt >12 && cnt <= 24
    fid2=fopen('output2.txt','a');  %output2 contains the strain values for 100ksi steel
    fprintf(fid2,'%.5f
',estrain100_1);
end
if cnt > 24 && cnt <= 36
    fid3=fopen('output3.txt','a'); %output3 contains the strain values for 100ksi steel (tension controlled) with phoprime = rho
    fprintf(fid3,'%.5f
',es\nstrain100_1);
end
if cnt > 36 && cnt <= 48
    fid4=fopen('output4.txt','a'); %output4 contains the strain values for 100ksi steel (compression controlled) with phoprime = 0
    fprintf(fid4,'%.5f
',estrain100_1);
end
if cnt > 48 && cnt <= 60
    fid5=fopen('output5.txt','a'); %output5 contains the strain values for 100ksi steel (compression controlled) with phoprime = 0.5*rho
    fprintf(fid5,'%.5f
',estrain100_1);
end
if cnt > 60 && cnt <= 72
    fid6=fopen('output6.txt','a'); %output6 contains the strain values for 100ksi steel (compression controlled) with phoprime = rho
    fprintf(fid6,'%.5f
',estrain100_1);
end
end
fclose(fid)
fclose(fid1)
fclose(fid2)
fclose(fid3)
fclose(fid4)
fclose(fid5)
fclose(fid6)

output_plot

E.2 Input File

4 0.85 0.85 0.0 0.0 0.005
5 0.80 0.85 0.0 0.0 0.005
6 0.75 0.85 0.0 0.0 0.005
7 0.70 0.85 0.0 0.0 0.005
8 0.65 0.85 0.0 0.0 0.005
9 0.65 0.85 0.0 0.0 0.005
10 0.65 0.85 0.0 0.0 0.005
11 0.65 0.83 0.0 0.0 0.005
12 0.65 0.81 0.0 0.0 0.005
13 0.65 0.79 0.0 0.0 0.005
14 0.65 0.77 0.0 0.0 0.005
15 0.65 0.75 0.0 0.0 0.005
4 0.85 0.85 0.5 0.1 0.005
5 0.80 0.85 0.5 0.1 0.005
6 0.75 0.85 0.5 0.1 0.005
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APPENDIX F: STRAIN DATA FOR SECTIONS WITH OVERSIZED FLANGES

Table F-1 shows the minimum strain that could be achieved in a BT-72 section. From the data, it can be seen that the minimum value of extreme tensile strain for the case with deck is appreciably larger than its counterpart without deck, and the minimum value of strain is greater than the 0.005 strain limit. Therefore, this section will always be considered a tension-control member. In contrast, in most cases the member without the deck is able to reach the compression-control strain limit of less than the 0.002, and depending on the reinforcing can be a compression, tension, or transition zone type member.

Table F-1 Minimum Extreme Tensile Strains in Sections with Decks

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<tr>
<th>Member</th>
<th>$f'_c$ (ksi)</th>
<th>Minimum Strain with 108&quot; Deck</th>
<th>Minimum Strain with 144&quot; Deck</th>
<th>Minimum Strain without Deck</th>
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*At concrete strain = 0.003, the strain in the extreme prestressing strand was in compression