THE IMPACT OF CODE EVOLUTION ON DESIGN OF BRIDGE PIERS

Bridge inspection data released by the Federal Highway Administration and other agencies routinely indicate that nearly 10 percent of our nation's bridges are structurally deficient. Significant maintenance, rehabilitation, and or replacement of structurally deficient bridges require enormous budgets and periodical or permanent closures for critical traffic. Structural engineers make difficult and costly decision on a regular basis to maintain bridge safety, economy, and operability.

Over the past 35 years, concrete bridge pier design capacity under combined axial and flexural loads has been impacted by the code evolutions. In this article, the key changes are highlighted to better inform bridge and building structural engineers as they support and influence key infrastructure decisions at the national and local levels.

For example, ACI 318 and by Reference ASCE 7 incorporated the Unified Design Provisions in the main body of the code in 2002 where a concrete section is defined as either compression-controlled or tension-controlled, depending on the magnitude of the net tensile strain in the reinforcement closest to the tension face of a member. The changes included revisions to capacity reduction factors, revisions to load factors, and a major redefinition of the transition zone between compression-controlled to tension-controlled limits.

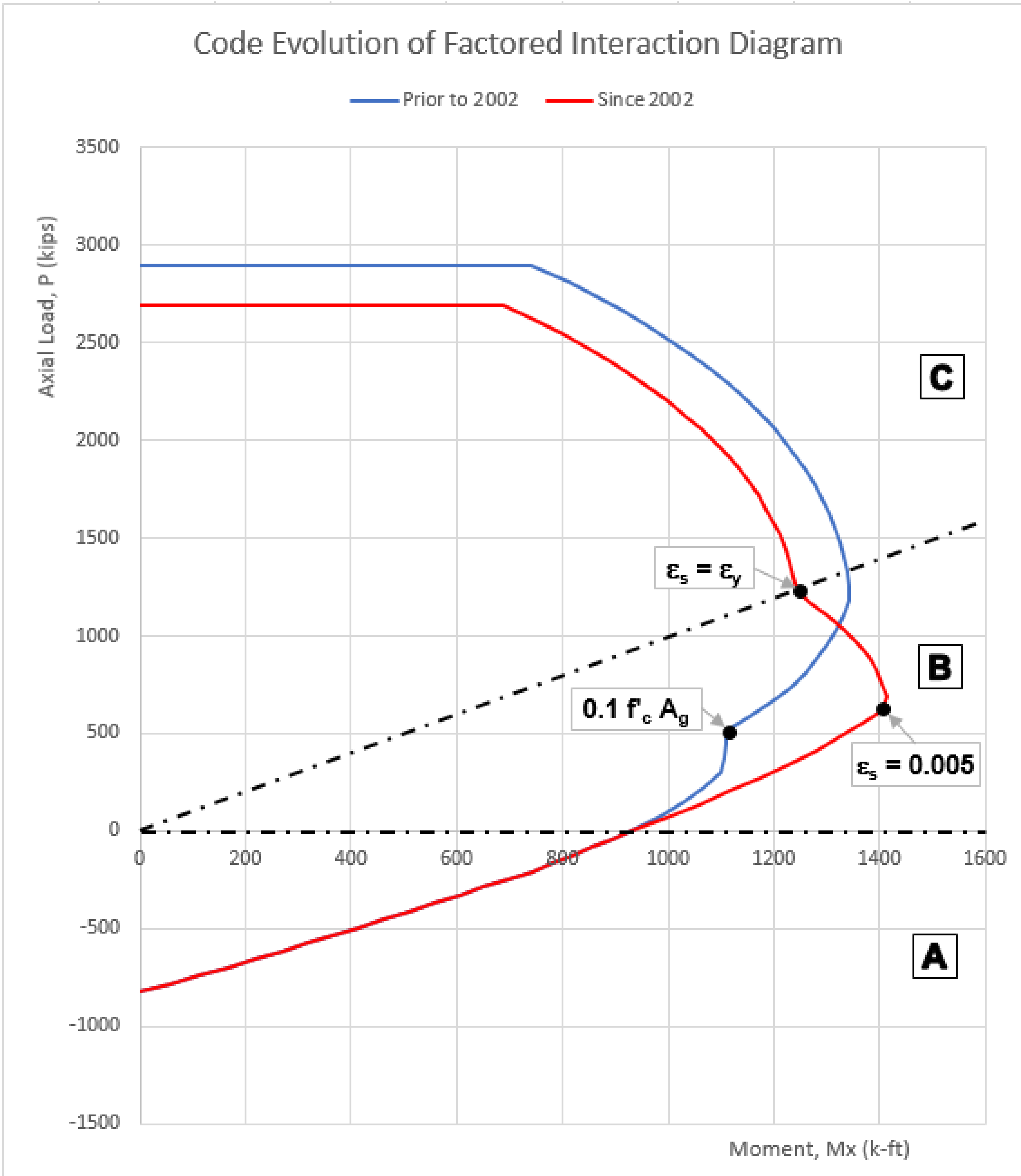
For a typical bridge pier section, two interaction diagrams are superimposed to compare and contrast capacity before and after the introduction of major code changes at the beginning of the century. The following observations can be made from the figure below and are designated as A, B, and C on the figure:

A. Pure Tension to Pure Bending: From pure tension to pure bending (axial load = 0), the ϕ factor remained as 0.90 despite the reduction of load factors which reduces the strength requirement by about 10 percent (see Table below) for newer code editions. For the evaluation of an existing design, the reduction of load factors affords an additional 10 percent margin for possible loading modifications. However, at the pure tension to pure bending ranges, this added capacity may not be very useful as most bridge sections are not governed by axial tension loads.

Reduced Required Strength since 2002 Code	
Live Load = 0.50 * Dead Load	11%
Live Load = 1.00 * Dead Load	10%
Live Load = 1.50 * Dead Load	9%

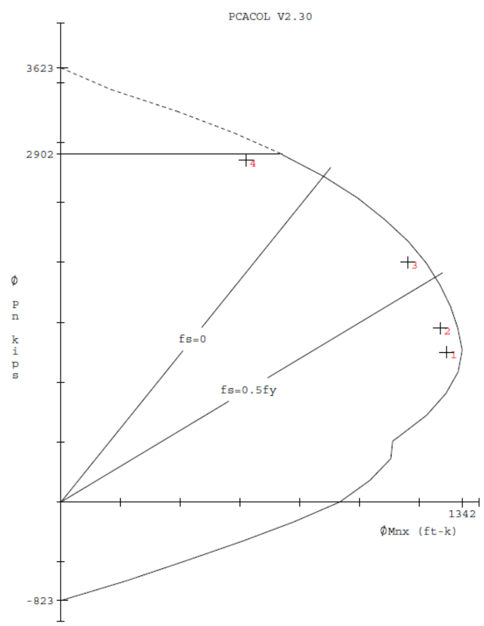
B. Pure Bending to Compression-Controlled Section Limit, (balanced point, $\varepsilon_s = \varepsilon_y$): Prior to 2002, the factor ϕ linearly transitioned from 0.90 at ϕP_n of zero (i.e. pure bending) to a factor ϕ of 0.70 (0.75 spiral) at the smaller of 0.10 $f'_c A_g$ or ϕP_n at balanced point ($\varepsilon_s = \varepsilon_y$). Since 2002, the factor ϕ transition changed to remain at 0.90 up to ε_s = 0.005 representing the new tension-controlled limit with a linear transition to the compression-controlled ϕ factor of 0.65 (0.70 spiral) at the balanced point where $\varepsilon_s = \varepsilon_y$. Existing piers design governed by 0.10 $f'_c A_g$ prior to 2002 code will have significant reserve capacity when investigated by later code in addition to 10 percent demand margin from load factor reduction.

C. Compression-Controlled Section Limit, (balanced point, $\varepsilon_s = \varepsilon_y$) to Maximum Compression Capacity Limit: In compression-controlled sections, the change of the ϕ factor from 0.70 (0.75 spiral) to 0.65 (0.70 spiral) may be offset by the corresponding reduction of load factors. This may not result in any significant design changes. It is interesting to note that the portion of the transition zone near the compression-controlled limit (within **B**) exhibits similar behavior.

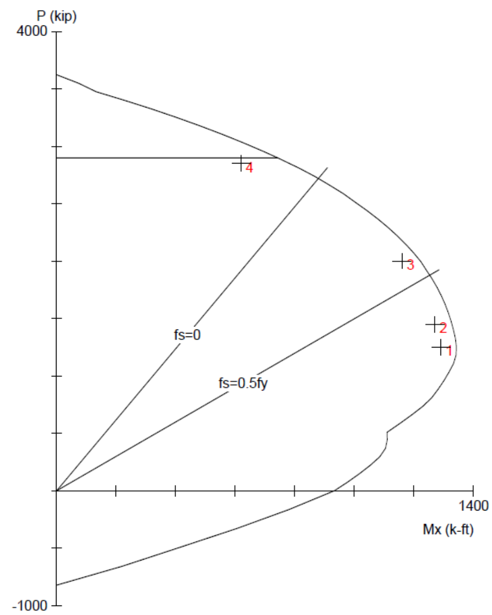


Interaction diagram results from pcaColumn/spColumn v3.00 (blue) and v5.50 (red)

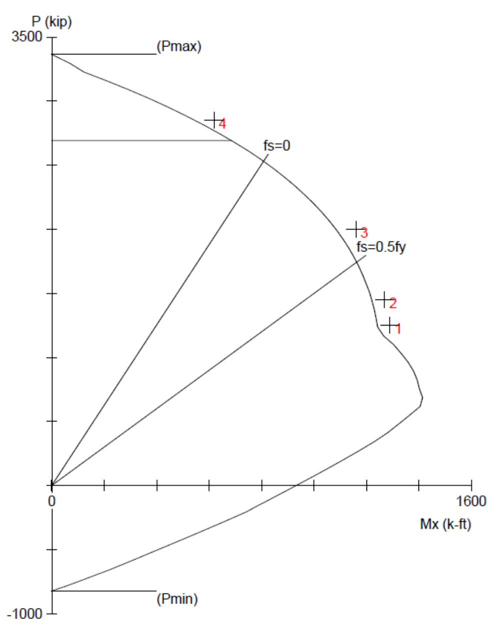
A set of factored design load points are considered to illustrate the observations above. In compression-controlled ranges, the bridge pier designed with factored design loads in the 80s and 90s would not meet the design requirements for 2000s and beyond if the same factored design loads are utilized. This is mainly due to the lowering of the capacity reduction factor for compression controlled sections from 0.70 (0.75 spiral) to 0.65 (0.70 spiral). However, a detailed examination of the service loads in conjunction with lower load factors may reveal the true and final impact on the design.



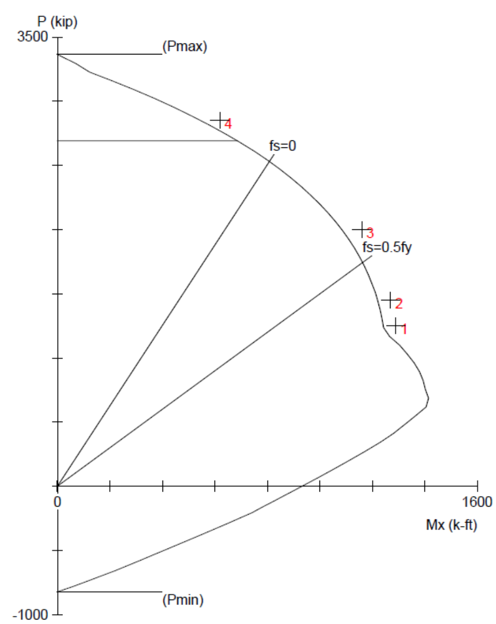
PCACOL v2.30 - Column Design 1980s



PCACOL v3.00 Column Design 1990s

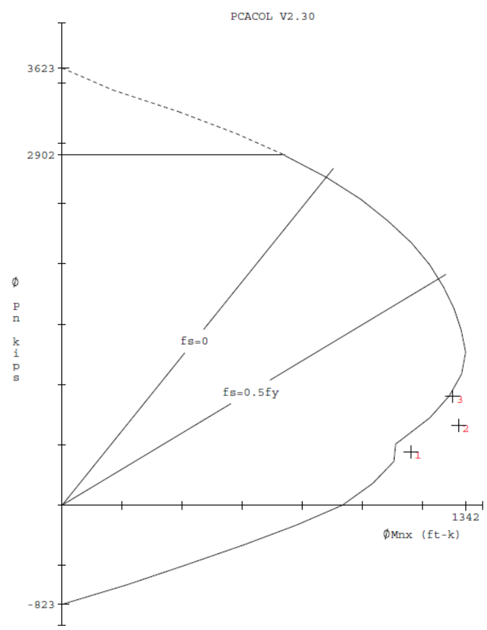


pcaColumn v4.10 - Column Design 2000s

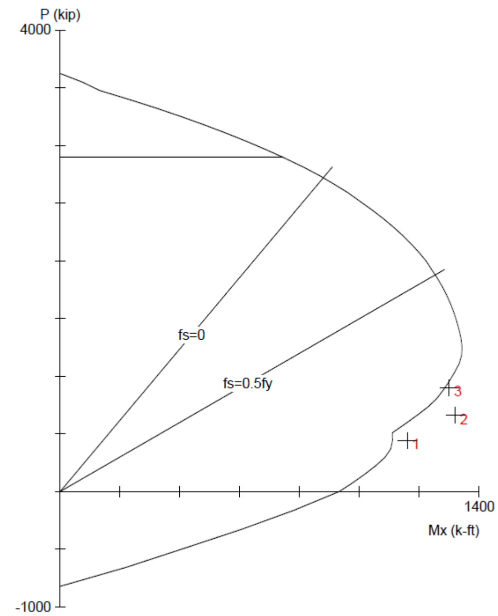


spColumn v5.50 - Column Design 2010s

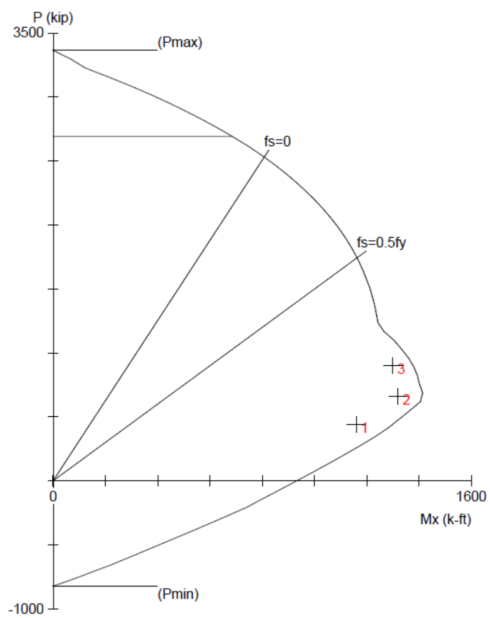
Another set of factored design load points are considered to illustrate the observations above. In transition and tension-controlled ranges, the bridge pier designed in the 80s and 90s showing deficiency in capacity would meet the design requirements for 2000s and beyond. The earlier designs are conservative due to their application of compression-controlled capacity factor of 0.70 in these ranges. The figures below indicate that the codes evolved to relax the conservatism on flexural capacities between zero axial load levels (pure bending) and 0.10 $f'_c A_g$. This can be observed by the disappearance of the pinch in the interaction diagram at 0.10 $f'_c A_g$ axial load level. Load point 1, 2, and 3 are now well within the interaction diagram without any additional revisions to the load factors that could further lower the strength requirements.



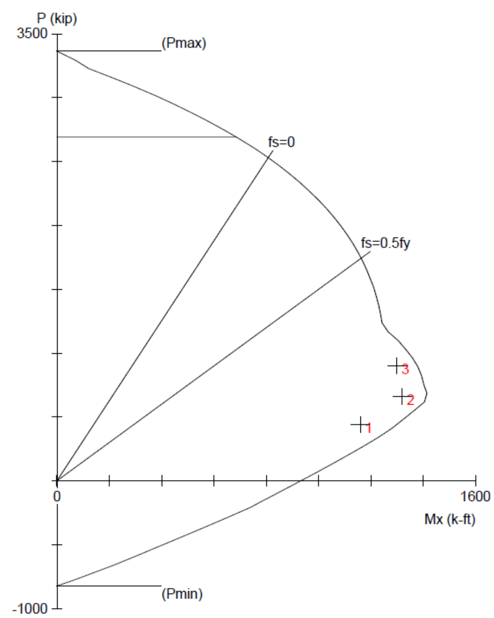
PCACOL v2.30 - Column Design 1980s



PCACOL v3.00 Column Design 1990s



pcaColumn v4.10 - Column Design 2000s



spColumn v5.50 - Column Design 2010s

Conclusions

Key changes in the design of concrete compression members subject to combined axial and flexural loading introduced early 2000s as summarized above may need to be carefully considered in making detailed assessments of the bridge pier or building column sections. Structural designers may examine the interaction of revised capacity reduction factors and load factors to optimize newly designed sections or properly investigate and assess the condition of existing building and transportation structures.

Additional capacity in section design where applicable can be employed to eliminate unnecessary retrofit or strengthening of building and bridge structural members. Similarly, the reduced capacity in section design where applicable needs to be properly assessed in order to determine the section adequacy to preserve funds and save lives.

References

[1] Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14), American Concrete Institute, 2014
[2] Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02), American Concrete Institute, 2002
[3] Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95), American Concrete Institute, 1995
[4] Building Code Requirements for Reinforced Concrete (ACI 318-83) and Commentary (ACI 318R-83), American Concrete Institute, 1983
[5] spColumn v5.50 – Design and Investigation of Reinforced Concrete Sections subject to axial and flexural loads, StructurePoint LLC., 2016
[6] pcaColumn v4.10 – Design and Investigation of Reinforced Concrete Sections subject to axial and flexural loads, StructurePoint LLC., 2008
[7] PCACOL v3.00 – Design and Investigation of Reinforced Concrete Column Sections, Portland Cement Association, 1999
[8] PCACOL v2.30 – Design and Investigation of Reinforced Concrete Column Sections, Portland Cement Association, 1994