Practical Analysis with the AISC 13th Edition

Emphasizing the use of the RISA Building System

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1.0 Introduction

The 13th edition of the AISC manual includes a new specification for structural steel buildings (AISC 360-05). This specification contains a number of changes that have caused some confusion and concern among many practicing structural engineers. While there is a rational basis behind each of these provisions, that basis is not always clear. What's more is that some of these provisions will force fundamental changes in the day to day practice of structural engineering. Therefore, the primary purpose of this document is to bridge the gap between the academic theory as presented in the code provisions and practical methods of implementing them in structural engineering design.

The provisions discussed in this paper will be limited to those directly related to computer analysis and design of structures, specifically commercial building structures. The specific focus will be directed towards the Chapter C requirements for Stability Analysis and Design as well as the practical methods of implementing these requirements.

1.1 General Requirements of Chapter C

When first read, Chapter C may be difficult to understand due to poor organization and presentation. There are just too many methods, alternate methods and exceptions presented. However, when these provisions and alternate methods are separated out and organized in a sensible manner, the provisions should become significantly easier to understand. Thankfully AISC has provided us with Appendix 7: The Direct Analysis Method as an example of how they want these requirements to be implemented.

Chapter C and Appendix 7 are primarily concerned with the following issues:

1. Accounting for the effects of initial geometric imperfections.
2. Accounting for the secondary effects caused by story drift or member deflection.
3. Accounting for the material non-linearity due to residual stresses.

The list above is, of course, a simplification of the chapter C requirements as is intended only to summarize the new / changed provisions. Obviously, there are still requirements stating that the effects of flexural, shear and axial deformations must be considered. Since that is typically accomplished by the same sort of analysis methods that engineers have been using for 30+ years the discussion of those portions of chapter C is deemed unnecessary.

In addition, there are miscellaneous analysis issues that are largely neglected (foundation settlement, uneven column shortening), but which are mentioned in the commentary to justify some over-conservativeness of the provisions.

2.0 Geometric Imperfections

Beams and columns that are assumed to be perfectly straight always have some natural curvature to them, even if it's slight. Furthermore, during the erection of steel buildings there will often be some measurable out-of-plumbness of the columns due to imperfect construction.

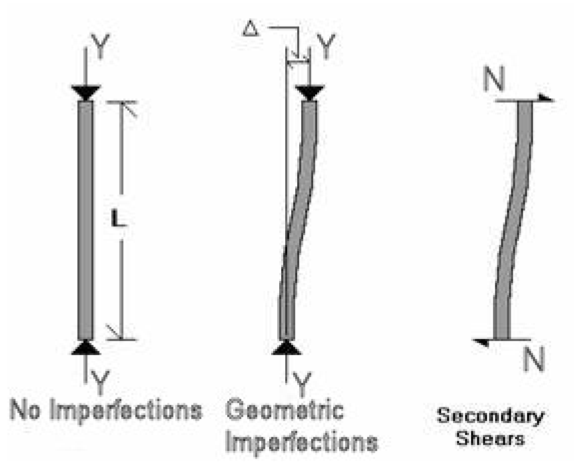
2.1 Historical Treatment

Historically, geometric imperfections have always been included in the design code by including a reduction in the allowable stress or column capacity. That's actually still the case, as the column capacity equations E3-2 and E3-3 are still based on an assumed out-of-straightness of L/1500. These equations were originally constructed assuming that the engineers would use a K factor to adjust the slenderness of their columns. This K factor was used to account for a number of issues, including additional geometric imperfections.

While there isn't anything wrong with this approach, the new edition of the AISC specification has added additional requirements that account for an additional assumed out-of-plumbness of L/500. Since this additional out-of-plumbness is not considered in the capacity equations, it must now be considered in the analysis or loading of the structure. When these effects are considered directly in the analysis, the new codes allow the engineer to enter the capacity equations using a K factor of 1.0.

2.2 Direct Analysis Method (Notional Loads)

The Direction Analysis Method (appendix 7) uses notional loads to account for the additional L/500 out-of-plumbness required by the specification. In section 7.3, a Notional Load equal to 0.2% of the gravity loads is required to be applied at each floor level. This force can easily be derived by introducing secondary shear forces to account for the destabilizing moment caused by an initial out-of-plumbness of 1/500 of the story height.



Where:

$$N \times L = Y \times \Delta$$

Solving for N when Delta is L/500, immediately leads to the N = 0.002 Y given in the Direct Analysis method. Since an assumed out-of-straightness of L/1500 is already included in the column capacity equations, there is inherent conservativeness built into this value. However, AISC may be making the argument that this L/1500 is to apply to the natural crown in individual column rather than to the overall out-of-plumbness of the structure.

2.2.1 When Notional Loads can be Reduced or Excluded

Because the MAXIMUM out-of-plumbness allowed by Code of Standard Practice for Steel Buildings and Bridges is L/500, the method allows the engineer to reduce these if a smaller out-of-plumbness can be justified. The reduced notional loads can be calculated just as easily using the previously derived equation when given an alternate out-of-plumbness ratio.

When second order deflections are less significant, the notional loads may be EXCLUDED for load combinations which include traditional lateral loads such as wind and seismic forces. However, if the second order deflections exceed 1.5 times the first order deflections, then these notional loads must be considered for ALL load combinations.

The rationale for this is that the effect of geometric imperfections has only a small effect on internal forces at low deflection levels. However, for slender structures that experience significant 2^{nd} order deflections the stability effects of these imperfections can become significant and should be considered.

2.3 Simplified Methods

The simplified methods also require a Notional Load equal to at least 2% of the gravity load. The main difference is that the simplified method requires you to apply this to ALL load combinations and the Direct Analysis Method may allow you to exclude the Notional Load from load combinations that already consider wind or seismic.

2.4 The Direct Analysis Method as Implemented by RISA

The RISA family of programs does not automatically account for geometric imperfections. However, RISA has introduced new Load Categories NL, NLX, NLY, and NLZ that the user can use to account for these loads on their own.

2.4.1 Further Automation of Notional Loads

The next release of the RISA Building System will include an automatic Notional Load generator where the program automatically calculates these notional loads based on a user defined gravity load combination.

Also included in the next release of RISA will be an improved Drift report that specifically identifies the ratio of 2nd order drift to 1st order drift for each story level. This can be used to easily determine when Notional Loads must be added into the Wind and Seismic load combinations.

3.0 Secondary Moments (P-Delta)

Chapter C of the AISC specification makes frequent comments about a second order analysis. By this, they mean any analysis that considers the geometric non-linearity of the structure response. They clarify this numerous times to indicate that any analysis that considers both P-Delta and P-little delta satisfies that portion of the provisions.

Typical structural engineering software is very good at modeling the P-Delta effect, but is less capable of modeling the P-little Delta effect. For this reason, it becomes an issue of practical importance to understand the difference between those two effects and when they will be significant.

3.1 Historical Treatment

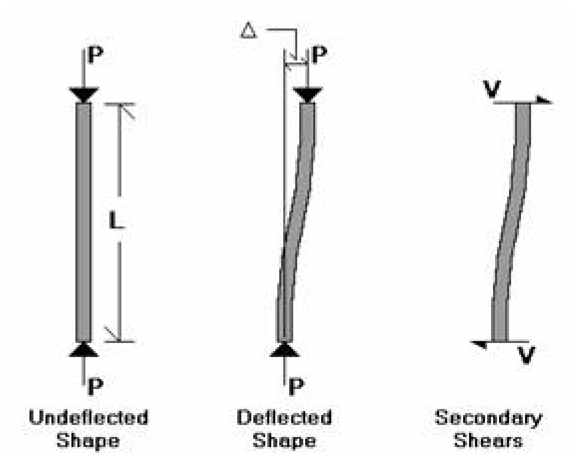
Second order effects were largely ignored in structural analysis until 1961 when the familiar $C_m / (1-f_a/F_e)$ equation was introduced into the AISC specification. This was considered adequate for the next 25 years or so.

Eventually, the $B_1M_{nt}+B_2M_{lt}$ equation was introduced into the code and commentary with the LRFD codes. At the same time, the LRFD codes specifically required the consideration of P-Delta. However, the LRFD specifications made no distinction between P-Big Delta and P-Little Delta. The requirement for considering P-Little Delta was added in the new 2005 AISC specification.

3.2 The Direct Analysis Method (P-Big Delta)

The P-Big Delta effect is essentially the destabilizing effect of the story drift on the axially loaded columns. This can be accounted for by running a true 2nd order analysis. However, most commercial programs used for building type structures do not have a true 2nd order analysis. Instead, they use either method 1 or 2 described below.

1) Secondary Shear Method: The application of secondary shears as explained in the image below can adequately account for P-Delta effects in most situations. This is a very general procedure that can be applied to hand calculations or to a Finite Element analysis.

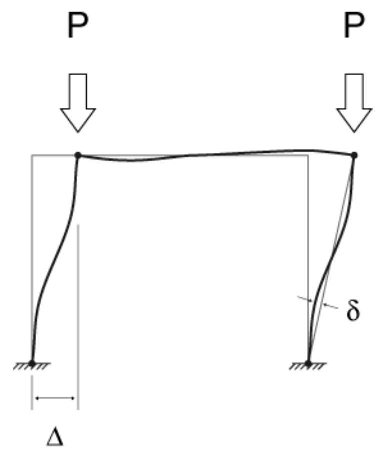


2) Geometric Stiffness Method: This method can be described as the softening effect of the lateral stiffness of the column to allow for additional lateral displacement. This method is particularly well suited for dynamic analysis as it will increase the natural period of the structure due to the softening of the lateral stiffness. This method is not as general and is only effective for structures where the lateral loads and the gravity loads are uncoupled.

3) True 2nd order analysis. There are programs out there that will perform a rigorous 2nd order analysis. But, these programs are normally geared towards extremely slender structures such as guyed towers. These types of analysis procedures are numerically very cumbersome because of the iterative nature of the technique. Essentially, these methods apply the load in small increments and adjust the stiffness matrix for each incremental deflection.

3.3 The Direct Analysis Method (P-Little Delta)

The P-Little Delta effect is essentially the destabilizing effect of individual member curvature on the axially loaded members. In practical applications, the effect of P-Little Delta will always be significantly smaller than the P-Big Delta effect. The following figure demonstrates the displacements that cause the P-Delta and P-Little Delta effect in a typical moment frame.

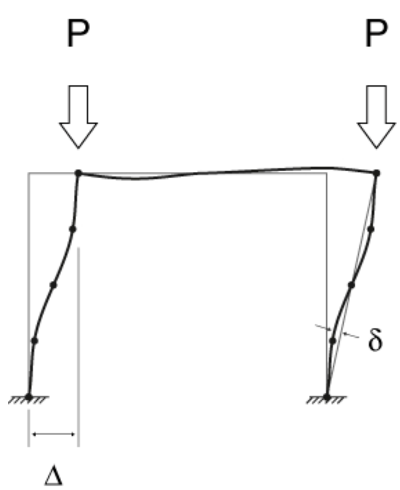


Even a cursory review of this figure should make it clear that P-Little Delta will have an effect that is an order of magnitude smaller than P-Big Delta for a normal moment framed structure.

3.3.1 Accounting for P-Little Delta

P-Little Delta can be accounted for by running a true 2nd order analysis. As mentioned earlier, most commercial programs used for building type structures do not have a true 2nd order analysis and rely instead on the Secondary Shear method or the Geometric Stiffness method for accounting for P-Delta. Unfortunately, neither of these methods directly account for the effect of P-Little Delta.

1) Secondary Shear Method: While the Secondary Shear method does not directly account for P-Little Delta, it can be easily adjusted to account for the effect as shown in the figure below.



Because the Secondary shear method is based entirely on nodal deflections, the introduction of nodes at the locations along the column where member displacement effects are at their maximum will adequately account for this effect.

2) Geometric Stiffness Method: This method actually can account for P-Little Delta. However, the typical software implementation of the method does not directly account for this effect. The typical implementation of the Geometric stiffness method would require either an iterative solution of the stiffness matrix, the addition of a mid-story column node between floor levels, or both. The addition of a mid height column node makes the practical implementation of P-Little Delta very similar to that described in item #1 above.

3) True 2nd order analysis: Some of the academic papers and presentations on the subject of P-Delta analysis will make a blanket statement such as, "*with today's computers and software, there is little reason not to do a rigorous 2nd*

order analysis for all structures." I cannot imagine a better statement to point out the naiveté of the academic community when it comes to the day to day practice of structural engineering in the United States. The fact of the matter is that none of the commonly used structural engineering programs in North America do a rigorous 2nd order analysis. Furthermore, doing so on a large scale model (20,000+ joints) would not be practical with the kind of computers in a typical engineering office.

3.3.2 Why You Can Ignore P-Little Delta

As mentioned earlier, there is an inherent disconnect between the academic community and practicing structural engineers. The academic community is very concerned with the fact that P-Little Delta is not considered in most structural analyses. Therefore, those members of the academic community that have been teaching the AISC stability seminar are becoming more and more vocal about requiring P-Little Delta in structural analysis in general.

However, even a careful study of the underlying papers and research that are referenced by the chapter C committee does not yield many practical examples of when P-Little delta will have a significant effect on the design of a typical building structure. The examples that are given are of extremely slender members subject to load approaching their elastic / Euler buckling loads. In industries such as communication towers where the structures are extremely slender, the design codes already require this type of analysis. But, this effect is unlikely to have a significant impact on a typical commercial building.

The Direct Analysis Method inherently acknowledges that P-Little Delta may not be important by stating that it may be neglected when, "the axial loads in all members whose *flexural* stiffnesses are considered to contribute to the lateral stability of the structure" are less than 15% of the Elastic / Euler buckling load of the member. This means P-Little Delta may be ignored for:

- All Beams (because their axial force will be less than 15% of the Euler Buckling Load)
- All Braces (because their flexural stiffness doesn't contribute to lateral stability)
- Gravity only Columns
- Lateral Columns that are NOT part of moment frames

Even when Direct Analysis Method would require you to include P-Little Delta, it is unlikely to have a significant effect on the overall analysis. Therefore, using some engineering judgment and the following arguments you could easily decide to ignore this effect for building type structures:

- The P-Little Delta deflection is an order of magnitude lower than the P-Big Delta deflection.
- If the deflection shape shown in section 3.3.1 is typical of a inter story column deflection, then there is a region where we have a positive P-Little Delta and a region with a negative P-Little Delta effect. These two effects will cancel each other out so that the Little Delta effect will not have any destabilizing effect on the overall structure.
- The Maximum moments in a column like this occur at the top and bottom of the column where there is no P-Little Delta effect. Therefore, the P-Little Delta is only serving to magnify moments in regions of the column that will not control the final code checking.

3.4 Simplified Methods

One of the "simplified" methods allows you to use the $B_1M_{nt}+B_2M_{lt}$ method to directly account for 2nd order effects. The code, in fact, considers this an adequate 2nd order analysis. Of course, there are practical limitations to using this procedure as it is cumbersome for hand calculations and impractical for automation into a Finite Element Analysis program.

Another one of the simplified methods allow you to partially bypass 2nd order calculations by increasing the notional loads on the structure from 0.2% to 0.42% and applying the B1 amplifier to all moments. However, this method is only allowed when the axial force in the member is less than 50% of the yield strength.

3.5 The Direct Analysis Method as Implemented by RISA

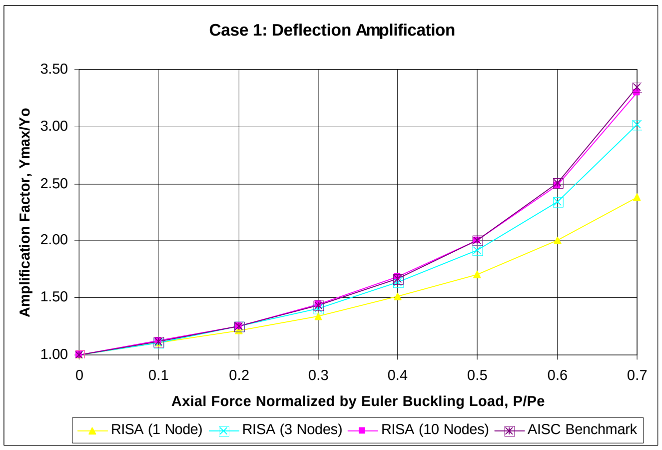
The RISA family of programs directly accounts for the effects of P-Big Delta whenever the user specifies that a P-Delta analysis should be included for a particular load combination. The program defaults require that a P-Delta analysis be performed in order to get code check results for all of the newer AISC codes.

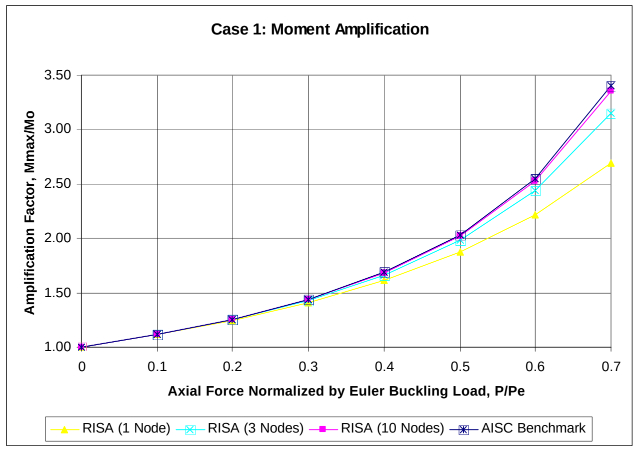
In addition, the programs go one step further and allow the user to include only the destabilizing effects of P-Delta that occur for compression members. Including P-Delta for members that are in tension actually has a stabilizing effect on the structure.

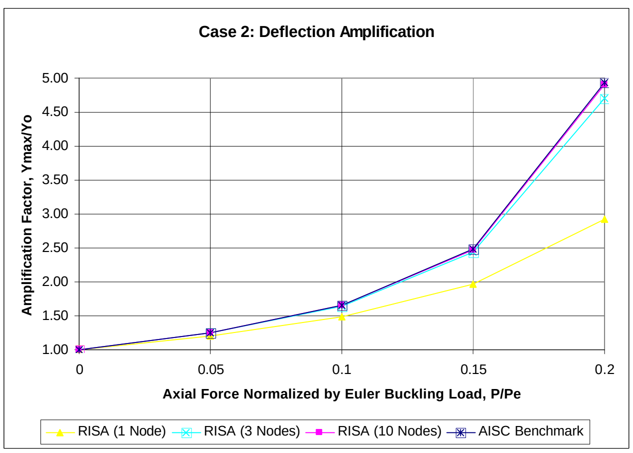
This allows the design engineer a further method of conservatism for structures where sway and second order deflections are a concern.

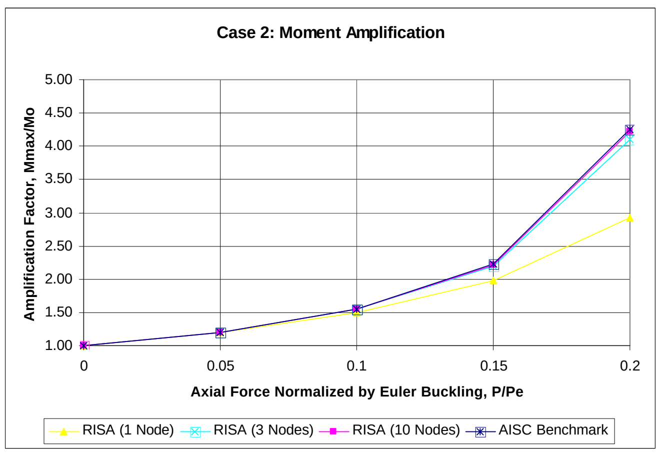
The effects of P-Little Delta are not directly considered in any of the most common Structural engineering analysis programs used in the United States, including RISA. However, the procedure discussed in section 3.3.1 will adequately account for P-Little Delta within RISA. This isn't as good as a rigorous 2nd order analysis, but it is close for all but the most slender structures.

The following charts summarize a comparison between the secondary shear method in RISA and a "true" 2nd order analysis by examining Case 1 and 2 of the Benchmark problems given in the commentary to Appendix 7. The RISA Method is accomplished by splitting the axial loaded members by adding nodes along the length of the axially loaded member.









4.0 Residual Stresses / Material Non-Linearity

Chapter C of the AISC specification requires that the analysis method used take into account the "stiffness reduction due to residual stresses on the stability of the structure and its elements". The concept here is that welding, differential cooling, cold bending, and cambering should introduce inherent stresses that will cause all members to experience some residual stresses even when they are not subject to any external load. These residual stresses then cause various portions of the cross section to yield earlier than the rest of the cross section. At ultimate level loads where the member is close to its plastic capacity, this may mean that the effective stiffness of the member is somewhat less than its initial linear stiffness.

4.1 Historical Treatment

The effects of residual stresses have been included in the column strength capacity curves used by AISC since the 1960's. Essentially, the codes have adopted a more conservative strength curve that compares well to the test results for weak axis buckling failure of wide flange shapes. Because of the residual stresses in the flange tips, weak axis buckling curves show a noticeably lower buckling stress than the strong axis curves.

While the column capacity curves have dealt with the issue of residual stresses for years, the code has never addressed the non-linear effect on the overall analysis of the structure. The rationale here was probably that an elastic analysis would be reasonable for the load levels which the structure would typically be subject to. This was a more accurate assumption for ASD service level loads than it was for LRFD loads.

4.2 Direct Analysis Method (Residual Stresses)

The Direct Analysis Method requires the use of a reduced Flexural Stiffness EI* for all members whose flexural stiffness is considered to contribute to the lateral stability of the structure. This would apply to all beams and columns that are part of lateral resisting moment frames.

$$EI^* = 0.8 t_b EI \qquad t_b = 1.0 \qquad \text{(when } a \frac{P_r}{P_y} \le 0.5\text{)}$$

$$t_b = 4\left[a \frac{P_r}{P_y}\left(1 - a \frac{P_r}{P_y}\right)\right] \qquad \text{(when } a \frac{P_r}{P_y} > 0.5\text{)}$$

Alternatively, a value of 0.8EI may be used for all members provided that an additional notional load equal to 0.1% of the gravity load is applied.

The Direct Analysis Method also requires a reduced axial stiffness EA* for all members whose axial stiffness is considered to contribute to the lateral stability of the structure. This would apply to all Columns and Braces that are part of the lateral force resisting system.

$$EA^* = 0.8EA$$

4.3 Simplified Methods

The simplified methods allow you to bypass the stiffness reduction due to residual stresses. The simplified 2nd order analysis has other restrictions requiring the re-introduction of K factors. Because the simplified first order analysis method neither accounts for the effect through K factors or through stiffness reduction, the method is only allowed for low axial stress levels where the stiffness reduction would be minimal.

4.4 The Direct Analysis Method as Implement by RISA

From a practical standpoint, this is probably the most challenging requirement to properly account for in the Direct Analysis Method. RISA's implementation of this the procedure has the following features:

- The ability to distinguish between members which contribute to the Lateral Stability of the model and those which are only there for Gravity loads.

- The ability to distinguish between Beams and Columns and Braces.
- The ability to distinguish between which members are part of moment (sway) frames.
- The ability to account for material non-linearity by adjusting the flexural stiffness of a member based on the axial load in that member.

The most challenging portion of new code implementation is usually accounting for new requirements in a way which does not overly complicate the interface. For years, the RISA interface has allowed for the assignment of Member Type (Beam, Column, Brace) and Member Function (Lateral vs Gravity). In addition, the RISA program has used “sway flags” for years to assist in the determination of K values and the Cm values used in the old ASD code. Therefore, these effects are easy to consider within the current interface of RISA-3D.

The last requirement (adjusting bending stiffness based on axial load) requires an iterative analysis. This can be time consuming for large models. Therefore, while RISA has fully implemented this provision, the program still gives the user the opportunity to turn off the stiffness adjustment if desired.

5.0 Analysis for ASD Load Combinations

Regardless of what the public relations people say, the 2005 AISC specification does not generally allow for ASD level design / analysis. There are numerous hoops that you have to jump through to actually be allowed to continue to use ASD design in the future.

5.1 Second Order Effects

The specification requires the use of ultimate level loads in your analysis. The rationale is likely that because 2nd order effects are non-linear, ASD design would have an advantage due to an inherently smaller impact from 2nd order amplification. To combat this, the code requires that all ASD analysis be done at 1.6 times the ASD Service Level loads. When comparing the demand forces to the allowable forces, the analysis results can then be divided by 1.6. This requirement effectively makes 2nd order effects higher for ASD analysis than for LRFD. How much ASD is getting penalized is hard to tell because of the non-linear nature of the effect. But, if P-Delta were to universally cause a 10% amplification for the structure’s moments and deflections, then the penalties would be as follows:

Dead Load 10% * (1.6 / 1.2) = 3.3%

Live Load 10% * (1.6 / 1.6) = no penalty

Seismic 10% * (1.6 / 1.4) = 1.4%

Wind 10% * (1.6 / 1.6) = no penalty

In addition, the ability to neglect P-Little Delta is not allowed for ASD load combinations unless the axial for in the member is less than 9.4% of the Euler Buckling Load as opposed to 15% for LRFD load combinations.

5.1.1 The RISA Implementation

The RISA Building system takes any Load Combination intended for the design per the ASD provisions of AISC 13th Edition and internally magnifies the load by this 1.6 factor as required in the specification. The forces and moments presented to the user, however, are then divided by the 1.6 factor again. This is done so that the user can rely on his analysis / design results without any manipulation required outside of the program.

5.2 Material Non-Linearity

The variation of flexural stiffnesses of the members also depends on the axial force in the member. Because ASD Load Combinations are done at Service Level Loads, the specification also penalizes them by a factor of 1.6. Again, this is an overly conservative penalty for most load combinations. The penalty is especially conservative for structures with mostly dead load and / or structures in a high seismic area.

5.2.1 The RISA Implementation

The RISA program adjusts the stiffnesses differently for ASD load combinations than it does for LRFD loads.

6.0 Conclusions

At first glance, the analysis requirements of Chapter C of the new 13th edition of the AISC manual may be confusing or intimidating. However, the Direct Analysis Method described in Appendix 7 is much more clearly presented. A close review of the Appendix and Chapter C will lead to the conclusion that the new requirements are primarily concerned with the following issues and their effect on stability of the structure:

1. Geometric imperfections.
2. Second Order effects caused by story drift or member deflection.
3. Material non-linearity due to residual stresses.

While the Direct Analysis Method makes the new requirements more understandable, it is complicated enough that it will likely eliminate the ability of an engineer to cost-effectively perform ANY hand calculations for steel structures. For that reason, it is essential that engineers understand these analysis provisions and acquire software that accounts for these effects such as the RISA Building System.