Effects of Nonverticality on Steel Framing Systems—Implications for Design

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The *Specification for Structural Steel Buildings* (AISC, 2005a), hereafter referred to as the *Specification*, allows the engineer a great deal of freedom in selecting the type of analysis to be used when assessing the strength and stability of a structure or framing system. Chapter C states:

> "Any method that considers the influence of second order effects (including P-Δ and P-δ effects), flexural, shear and axial deformations, geometric imperfections, and member stiffness reduction due to residual stress on the stability of the structure and its elements is permitted."

By individually identifying each of the phenomena that affect member and system strength in a framing system, the *Specification* is highlighting for the engineer the importance of each of these phenomena. Consequently, an understanding of each of these effects is beneficial in applying the new stability provisions of the *Specification*. This paper focuses on the effects of geometric imperfections, with an emphasis on frame nonverticality, or out-of-plumbness. The objectives of this paper are to:

- Illustrate how initial imperfections affect the strength of members and framing systems.
- Discuss how imperfections affect the magnitude and distribution of internal member forces and moments.
- Demonstrate the sensitivity of different types of unbraced frames to imperfections.
- Outline how imperfections are included in the AISC Specification approaches for assessment of frame stability.
- Discuss implications of the current limits on when imperfection effects may be neglected in the presence of lateral loads in the direct analysis approach of Appendix 7.

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The first section of the paper describes the types of initial imperfections typically considered in planar frame analysis and their effect on members and framing systems. This is followed by a discussion of how the effects of imperfections are treated in the AISC *Specification* (AISC, 2005a).

A parametric study is presented in which the sensitivity of framing systems to imperfection effects is investigated with respect to a number of parameters, including slenderness ratios, leaning load levels, gravity-to-lateral load ratios, and lateral frame stiffness, as measured by a second-order to first-order drift ratio. Understanding the types of frames that show sensitivity to imperfections provides a basis for determining when imperfections can have an influence on their inclusion in a design approach and decisions on when they might be neglected in the analysis.

In addition to the sensitivity study, a number of columns and simple frames are analyzed with and without imperfections using the direct analysis approach for assessing frame stability outlined in Appendix 7 of the AISC *Specification* (AISC, 2005a). The differences in the interaction checks for simple columns and frames are used to discuss the current limits on when imperfection effects may be neglected in the *Specification* in the presence of a higher lateral load.

INITIAL IMPERFECTIONS

In planar frame analysis, two types of imperfections affect strength and stability due to the amplification of the in-plane moments:

- Frame out-of-plumbness (or nonverticality) that may occur during erection, designated by Δ_0 in Figure 1.
- Member out-of-straightness which is a sweep of the member between the member ends that occurs during fabrication (one possible pattern is shown by δ_0 in Figure 1). It is typically considered as a single curvature sweep with the maximum imperfection at the midpoint of the member.

The AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005b) specifies the following tolerances for these two imperfections:

- Maximum member out of straightness, δ_0, of $L/1000$.
- For buildings less than 20 stories, out-of-plumbness, Δ_0, of $H/500$ in any shipping piece with a maximum lean of 1 in. towards the exterior or 2 in. towards the interior over the building height. Additional restrictions based on building height are imposed as shown in Figure 2.

Imperfections are considered in design because they amplify the moments in members when second-order effects are considered. The principal effect of an initial out-of-straightness on an individual member is the additional of an internal moment when the axial load, P, acts through the initial out-of-straightness, δ_0, as shown in Figure 1. This moment reduces the maximum axial capacity of a column. P-δ_0 moments can also impact adjoining members if their effect is to amplify the end moments. In many practical cases, this effect on end moments (when present) is minimal.

The initial out-of-plumbness (or nonverticality) also affects member strength, with the additional moment caused by the axial load, P, acting through the nonverticality, Δ_0. The P-Δ_0 moment also impacts the forces and moments in connecting elements, including connections, beams, base plates, slabs, etc. Figure 3 shows a column in which P-Δ_0 moments are transferred from the column to the adjoining beam (and, of course, the connection between them.)

INCLUSION OF INITIAL IMPERFECTIONS IN AISC SPECIFICATION BASED DESIGN APPROACHES

In the most recent version of the AISC *Specification* (2005a), significant changes were made to the way in which stability may be assessed in steel framing systems. In Chapter C, the *Specification* states that the required strengths for member design may be checked with member forces and moments obtained from one of the following analysis methods:

- Second-order analysis using nominal frame stiffness and a minimum lateral load of $0.002Y_i$, where Y_i is the gravity load applied at level *i.*
- Direct analysis, as outlined in Appendix 7 of the AISC *Specification*.
- First-order analysis, provided limitations on axial load levels are met, with an additional notional load, N_i, equal to

$$N_i = 2.1\left(\frac{\Delta}{L}\right)Y_i \tag{1}$$

where

Δ = first-order drift

L = story height

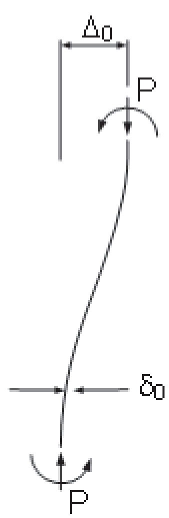

Fig. 1. Column flexure due to the axial load, P, *acting through imperfections* δ_0 *(out-of-straightness) and* Δ_0 *(out-of-plumbness).*

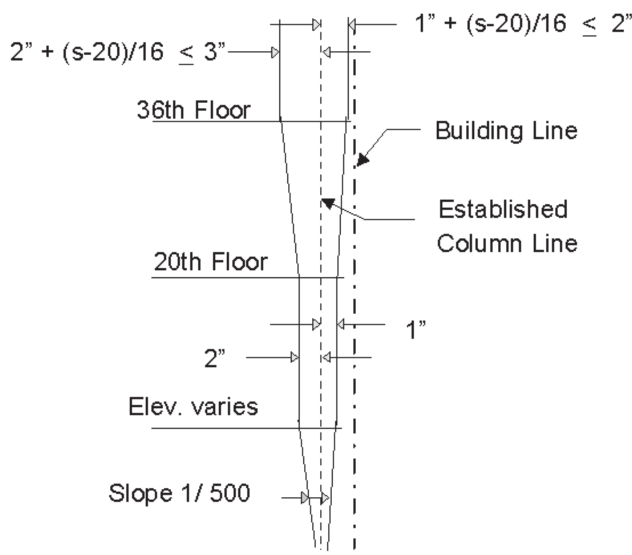

Fig. 2. AISC specified allowable erection tolerances for building frames.

In all cases, the initial out-of-plumbness is included, in some fashion, directly on the analysis side of the design process. A minimum lateral load of $0.002Y_i$, required in both the second-order and direct analysis approaches, provides amplified moments and forces equivalent to those obtained when an $L/500$ out-of-plumbness is modeled directly. In the first order approach, the effect of the initial imperfection is one factor included in the total notional load, N_i. The first-order approach is similar to notional load approaches used internationally, where a single horizontal load is used to account for multiple phenomena.

While out-of-straightness can have an important influence on the maximum strength of members in which the strength limit involves a nonsway failure mode, the modeling of member out-of-straightness within an analysis of the overall structural system is more cumbersome than the modeling of a uniform frame nonverticality. In lieu of direct modeling, the effect of out-of-straightness on the strength is accounted for in the axial strength term of the interaction equation given by Equations E3-2 through E3-4 in the AISC *Specification* (AISC, 2005a). This is the case in all of the approaches listed above.

The direct analysis approach of Appendix 7 meets all of the criteria listed in Chapter C. In the development of the direct analysis approach (Maleck, 2001; Surovek-Maleck and White, 2004), an emphasis was placed on including in the analysis, in as transparent a fashion as possible, both the effects of geometric imperfections and the stiffness reduction due to inelasticity and residual stresses.

The direct analysis approach includes the effects of the two planar initial geometric imperfections as follows:

- Out-of-straightness is explicitly accounted for by either altering the frame geometry or adding a notional load of $N_i = 0.002Y_i$ at each level, where Y_i is the total factored gravity load acting on level i. The notional load is equivalent to a uniform nominal out-of-plumbness of $L/500$.

- Out-of-straightness is implicitly accounted for by calculating the compressive strength of the column using the column curve provided in Chapter E of the specification. The development of this curve includes the reduction in strength due to out-of-straightness.

By modeling the out-of-plumbness directly in the analysis, the increase in second-order moments due to the imperfections is incorporated in the analysis results. In contrast, the out-of-straightness is not measured directly as an increase in second-order moment, but is accounted for as a decrease in member strength.

FRAME SENSITIVITY TO INITIAL OUT-OF-PLUMBNESS

Structural engineers can intuitively understand that for some frames, such as highly redundant, laterally stiff frames, the initial imperfections will have a negligible effect on the overall frame strength. This naturally leads to the questions:

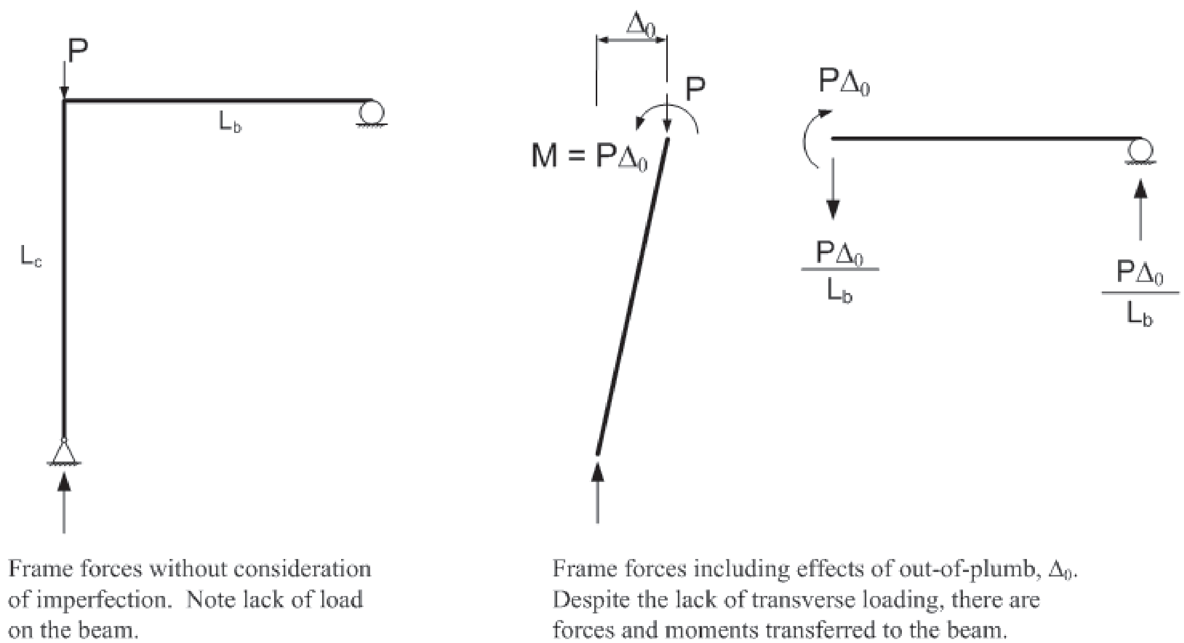

Fig. 3. Transfer of forces and moments due solely to initial nonverticality (out-of-plumbness).

"How sensitive are frames to initial imperfections?" and, "When can these effects be neglected?"

In order to address these questions, a parametric study was designed to determine:

- The sensitivity of different frame configurations to initial out-of-straightness.
- Which parameters most affect the significance of the out-of-plumbness on the strength of the frame.

Ultimate frame strength was determined using rigorous second-order, inelastic analyses (aka "plastic zone analyses"). The analysis included initial nonverticality, directly modeled in the frame geometry, as well as the effects of residual stresses on the inelastic response. This analysis approach is typically used to determine the "exact" strength of a frame. (Kanchanalai, 1977; White and Chen, 1993). Initial out-of-straightness was not modeled in order to isolate the effects of nonverticality on the frame response. Both P-Δ and P-δ effects were captured by the analysis. Frame parameters considered in the study included:

- Column slenderness ratios (L/r).
- Leaning load levels, that is, the axial load on pinned-pinned columns that do not provide stiffness to the lateral resistance of the frame.
- Gravity-load to lateral-load ratios.
- Lateral frame stiffness (as measured by the B_2 factor in the AISC *Specification*).

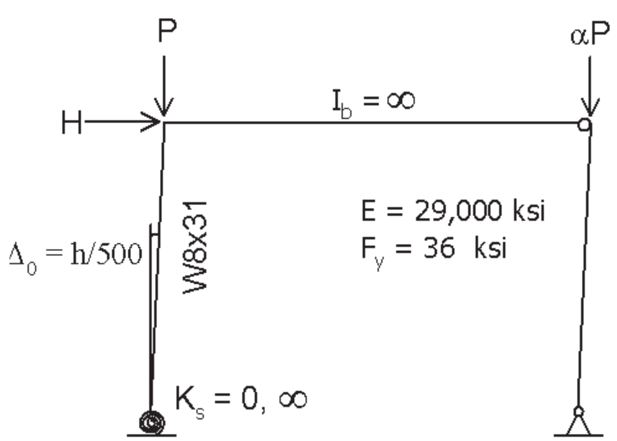

Fig. 4. Imperfection study portal frame example.

The frames studied were divided in two groups: axially loaded portal frames, shown in Figure 4, and multi-bay frames with distributed member loading. The behavior of portal frames with concentrated axial forces was examined with respect to the sensitivity of the ultimate frame strength to the inclusion of nonverticality in the model. In these studies, the beams were considered to be rigid and to remain elastic; thus, the load capacity of the portal frames was entirely dependent on the failure of the beam-column.

Previous studies that have been performed to assess the effects of imperfections (Wald, 1991; DeLuca, Faella and Mele, 1993; Clarke and Bridge, 1997) have considered frames that are loaded only with concentrated nodal loads. The effects of relative beam-to-column stiffness, inelastic behavior of beams, and particularly, moment transferred to the columns from the adjoining members due to transverse beam loading were not considered. Consequently, the studies are limited in that, in all cases, the load capacity of the frame was dependent only on the column strength. In addition to the portal frames, the frames considered in this study included 14 single-story, multi-bay frames as well as 11 multi-story, multi-bay frames with distributed beam loads. All of the frames (other than those originally studied by others) were designed per AISC strength criteria as well as to meet serviceability limits. The frames were designed considering a reasonable set of parameters with regard to frame stiffness and gravity-load to horizontal-load ratios, although a few very flexible frames were considered. Both strong-column, weak-beam and strong-beam, weak-column frames were considered to assess the impact of the failure mode of the frame on the imperfection sensitivity.

Portal Frame Studies

A number of small, stability critical portal frames in strong-axis bending were analyzed by second-order, inelastic analyses to assess the effects of nonverticality on sensitive benchmark frames. These types of frames are often studied to consider limiting cases with respect to beam-column and frame stability (Kanchanalai, 1977). The portal frame studied, which is oriented in strong-axis bending, is shown in Figure 4. Parameters considered in the study included gravity-load to horizontal-load ratios ($\Sigma P/\Sigma H$), as well as base fixity, column slenderness (L/r), and amount of leaning load (α). The pinned columns in frames UP and UR in Figure 9 represent leaning columns. The beam was modeled as rigid and of sufficient length so that overturning moments did not impact results. Values of the parameters considered included:

$\Sigma P/\Sigma H$ = 10, 20, 33, 50, 100, 200

L/r = 20, 30, 40, 50, 60, 70, 80, 90, 100

α = 0, 1, 2, 4

Figure 5 illustrates the impact of horizontal-to-gravity load ratios on imperfection sensitivity, measured as

$$\text{imperfection sensitivity} = [(\lambda_0 - \lambda_i)/\lambda_0](100\%) \quad (2)$$

where

λ_0 = ultimate load parameter for the perfect structure
λ_i = the corresponding parameter with the inclusion of an H/500 imperfection

In strong axis bending with a single leaning load, only those frames with $\Sigma P/\Sigma H > 50$ showed greater than 5% imperfection sensitivity. In Figure 6, the impact of leaning loads on imperfection sensitivity is highlighted. For frames with larger leaning loads (2P to 4P), the imperfection sensitivity is as high as 11%, but only when combined with very high gravity to horizontal load ratios. Column slenderness was not a significant parameter for lower values of $\Sigma P/\Sigma H$ but increased in significance for higher load ratios where the behavior is dominated by axial effects. As shown in Figure 6, the effects of leaning loads increases for higher $\Sigma P/\Sigma H$ again due to axial effects; however, this effect of higher leaning load diminishes as slenderness ratios increased. It is interesting to note that the slenderness ratio has little to no impact on the sensitivity of frames with low $\Sigma P/\Sigma H$ ratios.

Generalized Frame Studies

To extrapolate the results of the above portal frame studies to less idealized framing systems, 25 frames in strong axis bending were analyzed with and without imperfections to determine their sensitivity to the inclusion of imperfections in the geometry. The frames were subjected to distributed gravity loading. Fifteen frames were designed per AISC specifications, two were modifications of the first fifteen to illustrate a particular behavior, and eight were taken from previous research studies. The details on all of the studied frames are provided in Appendix A.

Single Story Frames

Twelve of the LRFD-designed frames were single-story frames ranging from one to eight bays. All of the frames, except one, were designed for a low 10 psf nominal wind load (to accentuate stability effects due to gravity load) and to meet a maximum drift criterion of H/500 [using a service load combination of $1.0D + 0.5L + 0.7W$ per Ellingwood (1996)]. The heavy gravity load was defined as 100 psf dead and 150 psf live, while light load was defined as 50 psf dead and 50 psf live. The results for the 12 single-story frames are presented in Table 1.

Only four of the frames considered had greater than 5% imperfection sensitivity. Of these, three were designed as weak-column/strong-beam frames in order to limit yielding in the beams at failure. In particular, two of these frames were designed with overstrength beams to limit or eliminate yielding in the beams at the failure load levels. All of these sensitive frames exhibited extensive spread of plasticity in the columns with no significant beam plasticity present at failure. The fourth frame exhibiting imperfection sensitivity was a two-bay, pinned-base frame that was allowed to exceed drift limits by 40% and had a second order amplification factor, B_2, of 1.9.

All of the frames that showed greater than 5% sensitivity to imperfections have B_2 values greater than 1.3. While Figure 7 shows a general trend in greater sensitivity with respect to higher B_2 values for the symmetric frames which meet drift criteria, it indicates that there is not a direct correlation between lateral frame stiffness, as measured by the B_2

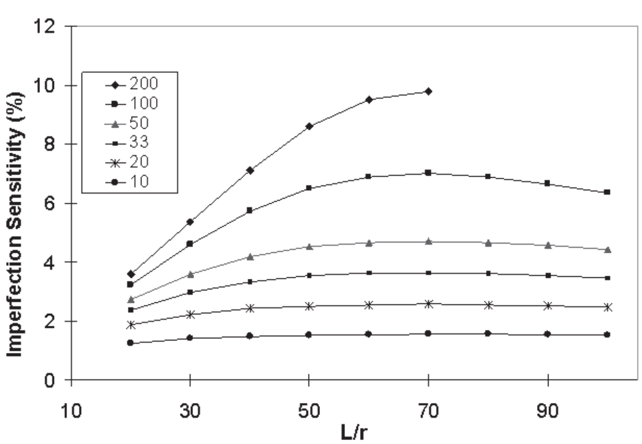

Fig. 5. Imperfection sensitivity versus ΣP/ΣH, strong-axis bending, $\alpha = 1$, $K_s = 0$.

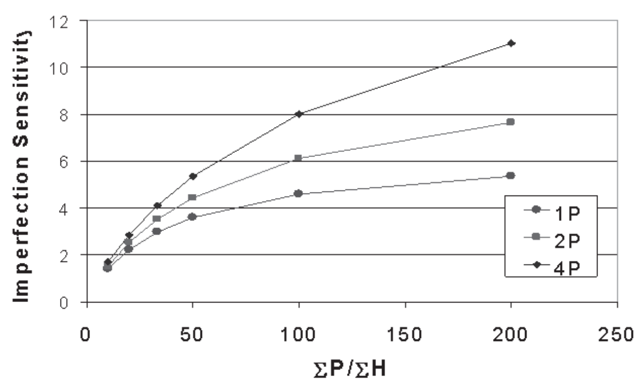

Fig. 6. Imperfection sensitivity due to leaning loads, L/r = 20, $K_s = 0$.

factor, and the effect of the initial imperfection on the ultimate strength of the frame.

The frames with higher B_2 values all exhibited limited yielding in the beams and failure largely associated with plasticity and eventually instability in the columns. In addition to the symmetric frames studied, two-bay, single-story unsymmetrical frames with differing base fixity were analyzed. Due to the tendency of these frames to drift under gravity load, the P-Δ_0 moments due to the imperfections were not significant when compared to the P–Δ moments due to the drift under gravity load, and neither exhibited a sensitivity greater than 1%, despite having B_2 values of 1.5 to 2.4.

Multi-Story Frames

Eleven multi-story frames were studied. Eight of these were two story frames previously studied by Ziemian (1990) with variation in the following parameters: symmetry, base fixity and load level. Of these frames, none exhibited greater than 2.25% imperfection sensitivity. Two six-story, two-bay fixed base frames were designed under heavy and light gravity loading. Design of the heavily loaded frame was controlled by the maximum live load combination, while design of the members in the lightly loaded frame was controlled by both wind and gravity load combinations. In both cases, imperfection sensitivity was less than 3%.

In general, only those frames that were "stability critical", that is those frames where the failure was dependent on instability of the columns rather than yielding of the beams, were likely to be sensitive to the initial imperfection. Leaning load levels and $\Sigma P/\Sigma H$ ratios have a direct impact on frame sensitivity, all other parameters remaining constant. While lateral stiffness has some impact on overall sensitivity of a frame to imperfections, there was no direct correlation.

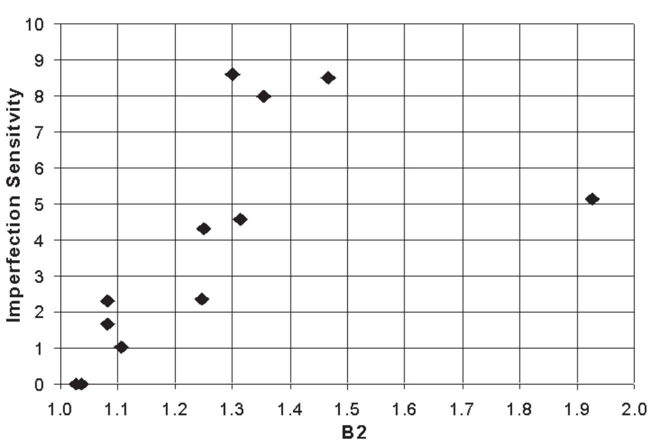

Fig. 7. Imperfection sensitivity for symmetric frames.

NEGLECTING THE IMPERFECTION EFFECT IN THE PRESENCE OF A LATERAL LOAD PER AISC APPENDIX 7

Appendix 7 of the AISC *Specification*, which outlines the direct analysis approach, allows for the notional load representing initial imperfections to be treated as a "minimum horizontal load" in frames where the ratio of second-order drift to first-order drift, Δ_2/Δ_1, (approximated by the B_2 factor) is less than 1.5. In other words, the notional load representing the imperfection may be neglected if a larger lateral load is applied to the structure. While not explicitly stated in the specification, it is presumed this limit must be checked and met at each story level.

Logic would suggest that since the notional loads represent physical imperfections, and since these physical imperfections would be present regardless of the load condition, the notional loads should be additive to other lateral loads. However, the verification studies used to validate the direct analysis method (Maleck and White, 2003) suggest that for stability critical frames that meet the B_2 limit, the unconservative error associated with disregarding the impact of the imperfection in the presence of a higher lateral load is typically less than 6%.

In order to verify the maximum unconservative error associated with neglecting the imperfection in the presence of a larger lateral load, a small verification study was run that focused specifically on this effect. The studies were designed to correspond with those used to verify the direct analysis method and included:

- individual column cases in which parameters considered fixity and gravity-load to horizontal-load ratios

- portal frames considered in the original verification studies for the direct analysis method that exhibited the highest unconservative error

In these studies, interaction diagrams were created based on results from rigorous second-order plastic zone analyses and compared to those developed from direct analysis results. The second-order analysis results for the direct analysis approach were performed using a closed form second-order analysis solution developed by LeMesurier (1977). LeMessurier's method accounts for both $P\Delta$ and $P\delta$ effects and is, for all practical purposes, exact for the sidesway problems studied in this work.

Both first-order (P versus M_1) and second-order (P versus M_2) interaction curves were developed, where M_1 is the maximum primary bending moment in the member due to the applied loading, and M_2 is the maximum internal second-order bending moment, located at the member ends in all the problems considered.

The P versus M_2 curves represent the strength of the member and, for the direct analysis approach, are the familiar interaction equations provided in Chapter H of the AISC *Specification*. The moments M_2 are the moments that must be transferred to the adjacent framing (in other words, to the beams and their connections at the beam-column joints) for satisfaction of equilibrium at the beam-column joints. The P versus M_1 interaction curves represent the maximum loadings that can be applied to the column or frames. The first order curves are of particular interest in design.

The error in the design interaction curves relative to the rigorous plastic zone solutions was measured for the columns and frames with and without imperfections. This error is defined as

$$e = \frac{r_{PZ} - r}{r_{PZ}} \tag{3}$$

where

r_{pz} = radial measure to the interaction curve developed using plastic zone analysis

r = radial measure to the interaction curve developed using the direct analysis approach

The radial error represents a measure of the error in the interaction check, or an overall error in the strength as measured. The horizontal error is a measure of the error in the first order column moment. The maximum moment in the cases studied occurred at the end of the column, so this represents the error in the moment being transferred to adjoining members and connections as well as the maximum moment in the column.

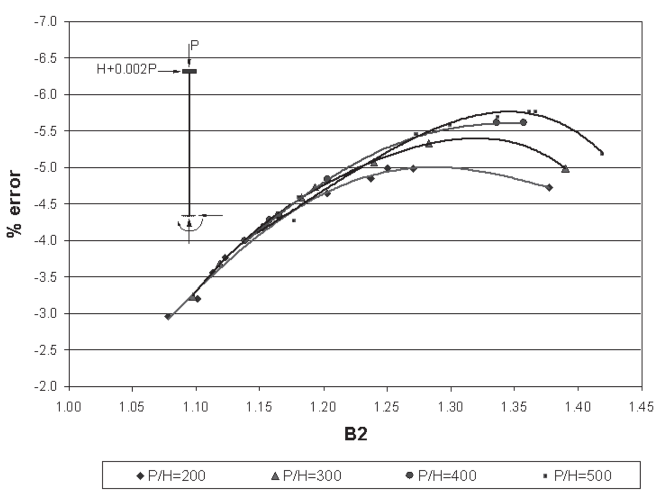

Fig. 8. Sensitivity of fixed-guided columns to exclusion of imperfection in addition to horizontal load, H.

Column Studies

As anticipated, the maximum strengths (as indicated by the interaction check) of columns with higher ratios of gravity to horizontal loads ($\Sigma P/H$) were more sensitive to the exclusion of the imperfection. For the fixed-guided case shown in Figure 8, the difference in strength between the columns with both the imperfection notional load and the lateral load and those that excluded the notional load was below 6% in all cases. It is interesting to note that the general trend does not continue to increase as lateral stiffness decreases. Similar error levels were observed for pinned-fixed columns and cantilever columns, and in no case did the unconservative error exceed 6%.

However, when considering the error in the moment calculated in the columns, the error was entirely dependent on the gravity-load to horizontal-load ratio. The error associated with the resulting first order column moment was as high as 50% for a $\Sigma P/\Sigma H$ ratio of 500. Because the error is associated with a high axial-load to horizontal-load ratio, the effect on the column interaction check is nominal (<6%), since the interaction check is dominated by the axial term.

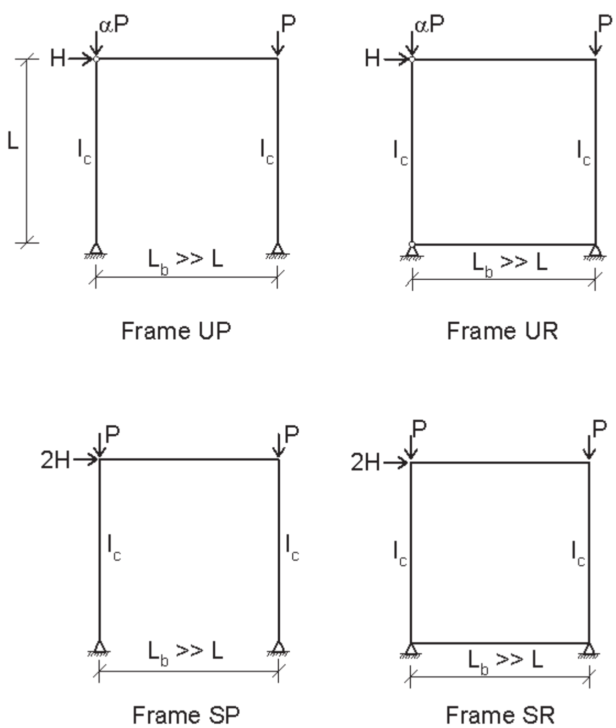

Fig. 9. Portal frames used in the verification of the direct analysis approach.

Portal Frame Studies

In this study, the most unconservative of the verification studies for the direct analysis approach (Maleck and White, 2003) were replicated with and without imperfections. Only cases where $B_2 < 1.5$ were considered. Both second-order analyses including P-δ effects and those including only P-Δ effects were performed. The verification frames are shown in Figure 9; they are similar in parameters to those shown in Figure 4 with the exception that the beam stiffness was also considered, as measured by the G factor found in the Commentary to Chapter C of the Specification, given by:

$$G = \frac{\sum \frac{I_C}{L_C}}{\sum \frac{I_G}{L_G}} \tag{4}$$

where

I_C	=	moment of inertia of the column
I_G	=	moment of inertia of the girder
L_C	=	unsupported length of the column
L_G	=	unsupported length of the girder

The frame designations indicate whether they are symmetric or unsymmetric (S, U), pinned or restrained at the base (P, R), oriented in weak or strong axis bending (W, S), have infinitely rigid (G0) or flexible beams (G1 or G3), and the level of leaning column load (α). Only the most critical frame studies were recreated (For complete results of the original study, the reader is directed to Maleck and White, 2003). The only variation from the initial study is that error was only measured for interaction values where $B_2 < 1.5$. The highest unconservative error was reported in the UP_W40_G1_α1 and SP_W60_G0 frames; both frames were laterally supported by columns in weak axis bending. None of the strong axis frames exhibited more than 6% error when a rigorous second-order analysis (that is, one considering both P-Δ and P-δ effects) was used. Figure 10 shows the first order interaction curves for the UP_W40_G1_α1 frame using a rigorous analysis, and Figure 11 shows the results for the P-Δ analysis. The complete results of the most sensitive cases are shown in Table 2.

The verification studies were based on the error in the column moment, only. For most practical adjoining members, if a distributed load is present, the moment being transferred by the column (particularly the moment due to an imperfection) will not be a significant portion of the maximum moment on the beam or connection. However, in the rare cases where the beam is not transversely loaded, such as the frame configuration shown in Figure 3, omission of the imperfection effect may lead to a rather unconservative design of the beam or connection. In cases where the design of connecting elements may be negatively affected by omission of this moment, such as in the case listed above, the notional load representing the imperfection should not be neglected regardless of the B_2 factor.

CONCLUSIONS AND RECOMMENDATIONS

For the engineer engaged in design of steel frames, a basic understanding of how initial imperfections affect frame strength and behavior is beneficial, particularly when interpreting the stability design provisions of the AISC Specification. The discussed studies of columns and framing systems considering the effects of nonverticality and a careful assessment of the current AISC provisions suggest the following:

1. The ultimate strength of most practical building frames will not be sensitive to initial nonverticality.

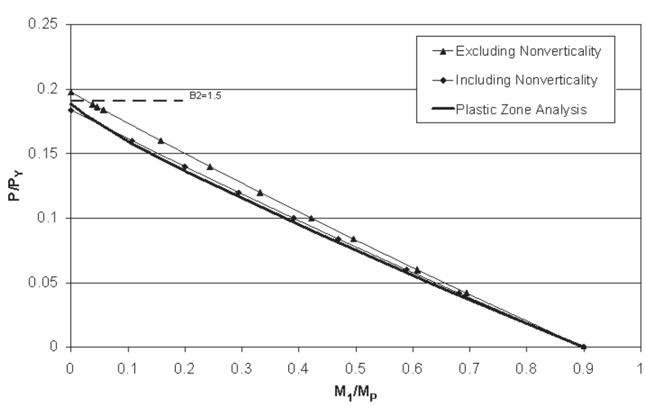

Fig. 10. First-order normalized interaction curves for UP_W40_G1_α1 frame P-δ *(rigorous) analysis.*

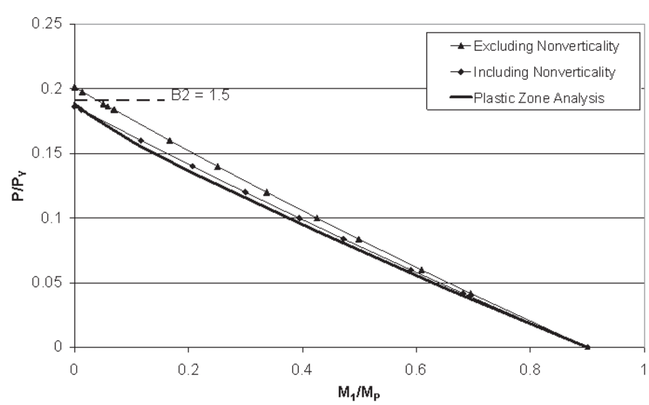

Fig. 11. First-order normalized interaction curves for UP_W40_G1_α1 frame, P-Δ *analysis.*

Table 1. Imperfection Sensitivity of Single-Story Symmetric Frames			
Imperfection Sensitivity	Number of frames	B_2	$\Sigma P/\Sigma H$
< 2%	4	1.03–1.1	60–280
2–5%	4	1.07–1.31	140–710
> 5%	4	1.3–1.93	280–1100

Table 2. Radial Error Measured in Interaction Checks for Verification Frames for B_2 < 1.5				
	Error with Imperfection		Error without Imperfection	
Frame Designation	P-Δ	P-δ	P-Δ	P-δ
SP S40 G0	-4%	-3%	-7%	-6%
SP S60 G0	-4%	-2%	-8%	-6%
SP S20 G0	-2%	-2%	-6%	-5%
UP W40 G1 α2	-1%	0%	-8%	-6%
SP W60 G0	-6%	-5%	-10%	-8%
SP W80 G3	-2%	-1%	-2%	-2%

2. The parameters that have the greatest effect on imperfection sensitivity include gravity-load to horizontal-load ratios, frame symmetry, and amount of leaning load. However, the primary cause of initial imperfection sensitivity in frames is the mode of failure and whether that failure is initiated by instability of a column rather than yielding in a beam.

3. Frames for which imperfection effects are negligible are not easily identified quantitatively, as no single parameter controls the sensitivity.

4. The provision of AISC *Specification* Appendix 7, in which the effects of imperfections may be neglected in lieu of higher lateral loads when B_2 < 1.5, is shown to produce a maximum unconservative error of 8%. This error occurred in a highly stability-critical portal frame laterally supported by a weak axis column only. When a less rigorous P-Δ analysis is used, the maximum unconservative error was 10%. For practical frames, this maximum unconservative error will be significantly smaller. Most practical frames will not be governed by the behavior of a weak-axis, unbraced, laterally resisting column.

5. If neglecting imperfections in the presence of a larger lateral load per Appendix 7, the engineer should be careful to consider the "special" cases where the beam design may be significantly impacted by the moment amplification due to the imperfection.

One point to consider is: When using the direct analysis approach in Appendix 7, is it easier to modify the geometry or calculate notional loads than to perform the calculations necessary to determine whether the imperfection effects can be neglected? Consider that if a direct second order analysis algorithm is used (that is, if B_1 and B_2 factors are not being separately calculated), a separate first-order analysis is still required to determine if the imperfection can be ignored. If the imperfection effect is small enough to neglect, inclusion of the notional load will have a negligible effect on the final design. In short, if the impact of including imperfections is negligible, economy will not be lost by including these effects, and this requires less effort than determining whether they can be neglected. It is also more rational, since the notional load represents a potential physical phenomenon that is independent of the load case.

Currently the maximum permissible imperfection is incorporated into the design provisions. There is a lack of data on measured nonverticality in constructed facilities to warrant a reduction of this imperfection. There are very few published studies that include surveyed measurements of a constructed building (Bridge, 1998; Beaulieu and Adams, 1978). Bridge concluded that many individual columns exceeded construction tolerances; however, story and global imperfection tolerances were met by compensating imperfections.

More data on erected structures would be useful if modifications to these provisions in the form of reduced requirements for imperfections were to be considered in future editions of the *Specification*.

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APPENDIX A

Parametric Study Frames

Beam and column designations used in the single and multistory frames are presented below. The frame designations are shown in Figure A.1. Steel designations used in the single and two-bay symmetric test frames are given in Tables A.1 and A.2, respectively. The remaining frame designs are given in Figures A.2–A.11. Table A.3 presents the B_2 factors, analysis results, and imperfection sensitivities for each of the test frames. The imperfection sensitivity was calculated with

$$\text{imperfection sensitivity} = [(\lambda_0 - \lambda_i)/\lambda_0](100\%)$$

where

λ_0 = ultimate load parameter for the perfect structure

λ_i = the corresponding parameter with the inclusion of an $H/500$ imperfection

Details of the frames with designations beginning with U or S may be found in Ziemian (1990).

For the symmetric frames:

story height = 12 ft
bay width = 25 ft
frame spacing = 25 ft

For the unsymmetric frames:

story height = 12 ft
bay widths = 20 ft and 30 ft
frame spacing = 25 ft

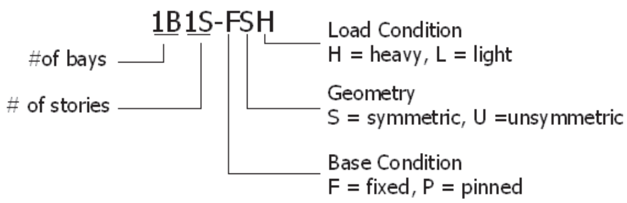


Fig. A.1. Key to frame designation.

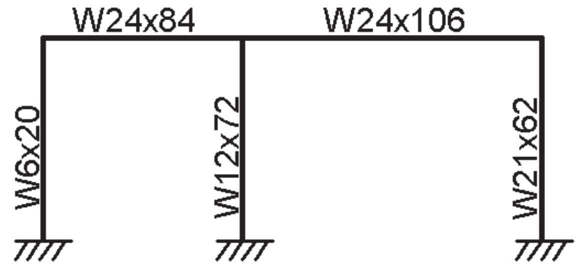


Fig. A.2. 2B1S-FUH.

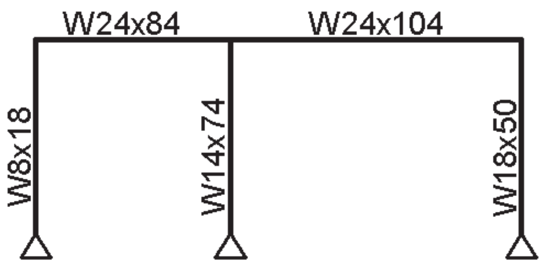


Fig. A.3. 2B1S-PUH.

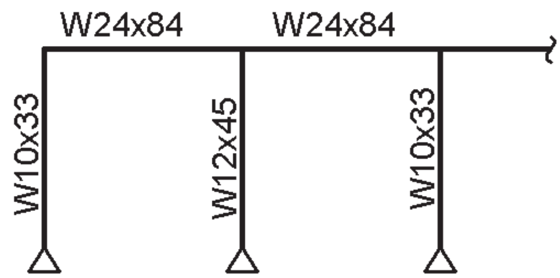


Fig. A.4. Frame 4B1S-PSH1.

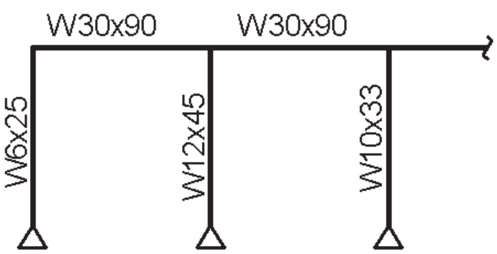


Fig. A.5. Frame 4B1S-PSH2 (oversized beams).

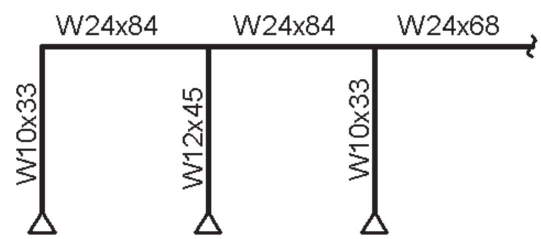


Fig. A.6. Frame 5B1S-PSH1.

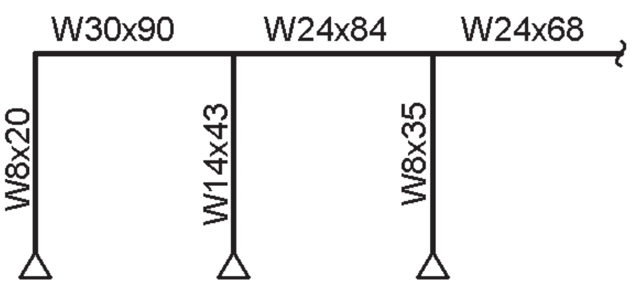


Fig. A.7. Frame 5B1S-PSH2 (oversized beams).

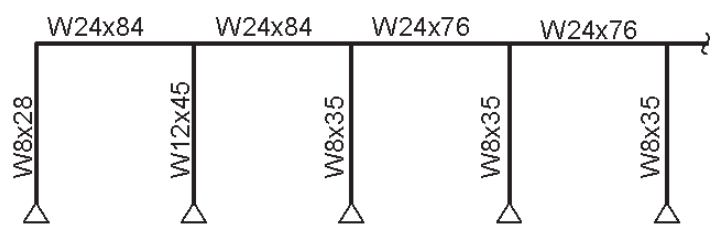
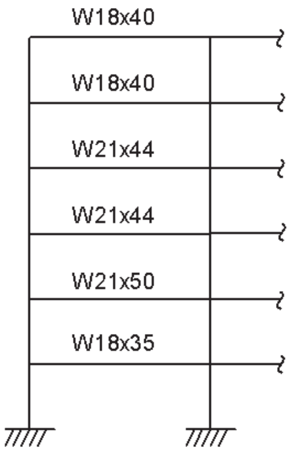
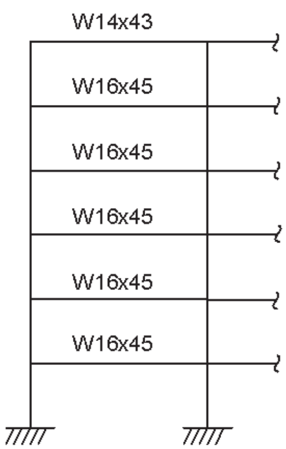


Fig. A.8. Frame 8B1S-PSH.



Story	Ext. Col	Int. Col
1	W14x99	W14x159
2	W14x99	W14x132
3	W14x82	W 12x96
4	W14x68	W12x72
5	W14x61	W12x45
6	W 8x28	W 6x15

Fig. A.9. Frame 2B6S-FSL.



Story	Ext. Col	Int. Col
1	W14x48	W14x74
2	W14x48	W12x58
3	W12x40	W 8x48
4	W10x39	W 8x35
5	W10x33	W 8x28
6	W 8x24	W 6x15

Fig. A.10. Frame 2B6S-FSL.

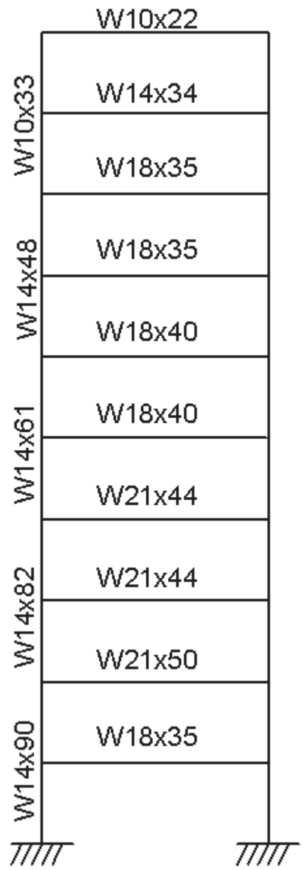


Fig. A.11. Frame 1B10S-FSL.

Table A.1. Single Bay Steel Sizes		
Designation	**Beam**	**Column**
1B1S-FSH	W30x90	W12x53
1B1S-PSH1	W30x99	W14x43
1B1S-PSH2	W24x104	W8x40
1B1S-FSL	W12x58	W8x31
1B1S-PSL	W12x58	W10x39

Table A.2. Two-Bay Symmetric Steel Sizes			
Designation	**Beam**	**Exterior Column**	**Interior Column**
2B1S-FSH	W27x84	W10x33	W8x40
2B1S-PSH	W24x94	W8x25	W8x40

Table A.3. Results of Planar Frame Analyses				
Frame Designation	B_2	λ_i	λ_o	**Sensitivity (%)**
1B1S-FSH	1.028	1.478	1.478	0
1B1S-PSH1	1.082	1.517	1.522	2.31
1B1S-PSH2	1.247	1.322	1.354	2.36
1B1S-FSL	1.037	1.165	1.165	0
1B1S-PSL	1.082	1.449	1.425	1.67
2B1S-FSH	1.106	1.606	1.623	1.11
2B1S-PSH	1.926	1.324	1.392	5.13
2B1S-FUH	1.546	1.485	1.486	0.02
2B1S-PUH	2.42	1.359	1.37	0.75
4B1S-PSH1	1.25	1.441	1.503	4.32
4B1S-PSH2	1.354	1.496	1.616	8
5B1S-PSH1	1.314	1.427	1.492	4.57
5B1S-PSH2	1.3	1.495	1.624	8.61
8B1S-PSH	1.47	1.351	1.466	7.84
2B6S-FSH	–	1.299	1.3	0.34
2B6S-FSL	–	1.413	1.418	0.37
1B10S-FSL	–	1.38	1.434	3.54
U-P36L	–	1.144	1.145	0.07
U-P36H	–	1.073	1.088	1.46
U-F36L	–	1.163	1.164	0.2
U-F36H	–	1.161	1.162	0.1
S-P36L	–	1.226	1.227	0.03
S-P36H	–	1.218	1.246	2.23
S-F36L	–	1.242	1.264	1.8
S-F66H	–	1.184	1.189	0.4