ON THE SYSTEM-BASED DESIGN FOR STEEL FRAMES USING INELASTIC ANALYSIS

Hao Zhang*, Haoyu Liu, and Kim J.R. Rasmussen
School of Civil Engineering, The University of Sydney, NSW, Australia
*Email: hao.zhang@sydney.edu.au

ABSTRACT

Design by inelastic analysis of overall system behaviour is permitted in several steel design specifications worldwide (e.g., the American Specification AISC360-10 and the Australian Specification AS4100-1998). Advanced inelastic analysis is better able to capture the system behavioural characteristics as they currently are understood. This paper presents a case study of the design of three planar steel structures using different design methods, including the Direct Analysis method in AISC360-10, the inelastic design method in AISC360-10, and the inelastic method ("advanced analysis") in AS4100. The effects of structural ductility (capacity of load redistribution) and failure modes on the design results are discussed.

KEYWORDS

Inelastic analysis, elastic analysis design, steel structures, nonlinear frame analysis, system-based design.

INTRODUCTION

The traditional steel design methods focus on the limit states of individual components and connections, based on elastic structural analysis. In the design process, the system effects are addressed only implicitly through the use of effective length factors. The design philosophy (except for seismic design) generally is based on a "first-hinge" approach, i.e., the strength limit state of the frame is defined when one or more members develops a fully plastic hinge. The effective length factor approach cannot predict the true interactions between members of a large structural system accurately. Moreover, the beneficial system effects such as load redistribution cannot be captured by elastic analyses. The recent increases in computing power and sophisticated inelastic structural analysis make it possible to predict the behaviour of complex steel structural systems quite accurately, including the effects of material nonlinearity (yielding), geometric nonlinearity, initial geometric imperfections, residual stresses etc.

With the advent of inelastic analysis, it is now possible to design a steel frame on the system level. The system strength can be directly assessed from the analysis without the need for checking the member-based design specification provisions. System-based inelastic design may also lead to more economic design than the first-hinge elastic approach by explicitly incorporating beneficial system effects such as load redistribution (Ziemian et al., 1992).

The most significant advantage of design by inelastic analysis, however, is that engineers can better understand the system behavioural characteristics. This advantage is especially important in the new paradigm of performance-based design, which is closely coupled to the issue of system behaviour. Since inelastic analysis can explicitly indicate the failure modes, it becomes possible to consider different performance objectives in design (e.g., first yielding or hinge, incipient system instability, etc.), and associate with each a specific reliability goal. Take the wind design for instance, the owner may choose "first hinge" or "collapse prevention" as the performance objective. The former is the current wind design objective. If the owner accepts a lower performance objective of "collapse prevention", the system is required to maintain sufficient stiffness to prevent collapse, but certain members (with adequate ductility) are permitted to enter the inelastic range. Such a design methodology is founded on rigorous nonlinear system analysis, which is no longer a formidable task from the structural analysis point of view.

The objective of this paper is to compare the designs of three planar streel structures using different design methods, including the Direct Analysis Method (DAM) in AISC360, and the two inelastic design methods in the American Specification AISC360-10 and the Australian Specification AS4100. The three methods are briefly introduced. The merits of system-based design by inelastic analysis are demonstrated through the examples. The roles of structural ductility (capacity of load redistribution) and failure modes on the design results are discussed.

DESIGN METHODS

Direct Analysis Method (AISC360-10)

The Direct Analysis Method in AISC360-10 requires a rigorous second-order elastic analysis with modelling of initial geometric imperfection (either by direct modelling or by applying notional loads) and application of a reduction factor of 0.8τ to the nominal elastic stiffness in structural analysis. Internal actions are obtained from the second-order elastic analysis and the strengths of components are checked using an effective length factor of unity for compression members. The limit state of in-plane flexural buckling is checked according to the interaction equation:

$$\frac{P_u}{\phi P_n}+\frac{8}{9}\left(\frac{M_u}{\phi M_n}\right)\leq 1.0, \text{when } \frac{P_u}{\phi P_n}\geq 0.2, \qquad (1)$$
$$\frac{P_u}{2\phi P_n}+\frac{M_u}{\phi M_n}\leq 1.0, \text{when } \frac{P_u}{\phi P_n}< 0.2,$$

where P_u and M_u are the required strengths (design actions), calculated from second-order analysis under the design loads; and P_n and M_n are the nominal compression and bending strengths, calculated in the plane of the frame. ϕ is the resistance factor for reliability consideration. With member and system instability being checked by the second-order analysis, the equation based design checks only need to be completed at the cross-sections. Compared to the traditional load and resistance design using the effective length procedure, the DAM greatly simplifies the assessment of frame stability by using the actual member length. However, the DAM still represents the member-based, “first hinge” design philosophy.

System-based design by inelastic analysis in AISC360-10

Appendix 1 of AISC360-10 allows, in principle, the use of inelastic analysis to supersede the member-based design equations in the Specification, provided that (1) the limit states covered by the Specification equations are detected by the inelastic analysis; (2) members and connections with elements subject to yielding have adequate ductility; and (3) a comparable or higher level of structural reliability is provided by the inelastic analysis than by member-based design. If a certain limit state cannot be modeled by the inelastic analysis, it needs to be verified by the corresponding Specification equation. To ensure adequate ductility of members and connections, two general methods can be used: (a) limiting the factors that affect the inelastic deformation capacity of components, such as the slenderness of cross-sectional elements and the unbraced length, and (b) limiting the inelastic deformation demands to be less than or equal to their (predefined) inelastic deformation capacities.

The AISC 360-10 approach to ensuring structural reliability is to reduce both the strength and stiffness of all members and connections by a factor of 0.9. The 0.9 factor on stiffness makes sure that the strength of a frame with slender members failing elastically will be factored downward. The AISC 360-10 commentary acknowledges that the factor of 0.9 has its origin in the AISC LRFD resistance factors of tension and flexural members that yield; its use in system-based design, although ‘‘deemed acceptable’’, is not based on system reliability analysis.

Design by inelastic analysis in AS4100-1998

The Australian Specification for steel structures AS4100 (1998) is one of the first specifications that permit the use of inelastic analysis to verify the integrity of structural systems. The cross-section must be compact and members must be fully braced against flexural-torsional buckling. One needs to check the integrity of the frame under factored loads, using (unfactored) nominal values of structural stiffness and strength. In addition to that, the section capacity and connection capacity need also to be checked according to the specification equations. For compact section, the section capacity check is given by:

$$\frac{P}{\phi P_y}+\frac{M_x}{\phi M_{px}}+\frac{M_y}{\phi M_{py}}\leq 1 \qquad (2)$$

in which ϕ = the member resistance factor (0.9 in AS4100), P = design axial force under factored loads, P_y = nominal axial yield strength. M_x and M_y are design bending moments about the major principal x-axis and minor principal y-axis, respectively, M_{px} and M_{py} are nominal section plastic moments about x-axis and y-axis, respectively. AS4100 allows the internal actions (P, M_x and M_y) to be determined by inelastic analysis but still requires section capacity to be checked by specifications. As will be seen later in this paper, the need for separate section capacity check negates efficiencies to be gained by using inelastic analysis for system-based structural design.

EXAMPLE STEEL FRAMES

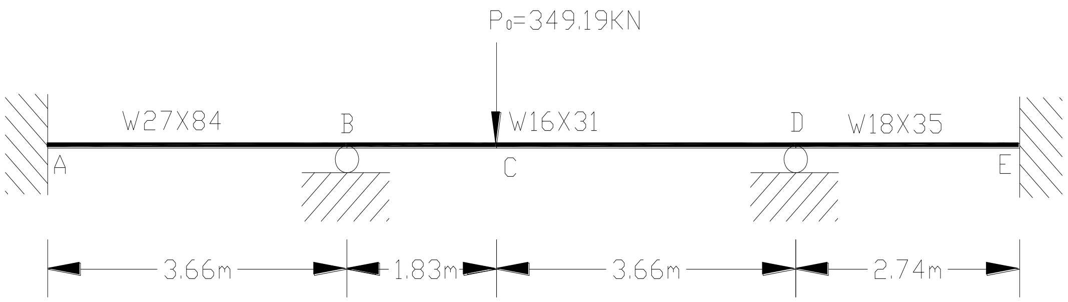


Figure 1 Example 1: 3-span continuous beam.

Three examples were adopted from the literature. The first example is a 3-span continuous beam as shown in Figure 1. This example was adopted from Ziemian and McGuire (2007). The nominal yield strength and modulus of elasticity for the members are 345 MPa and 200 GPa, respectively. The reference load $P_0 = 349.19$ kN.

The second example, shown in Figure 2, is a 2-bay, 2-storey unsymmetrical frame subjected to gravity load. The distributions of loads at different levels are as shown in the figure. The frame was adopted from (Ziemian et al., 1992). The nominal yield stress and modulus of elasticity for the members are 248 MPa and 200 GPa, respectively. The frame has an initial out-of-plumbness of 1/500 towards the right. The reference load P_0 is 147.11kN/m.

Example 3 is a portal frame adopted from Ziemian and McGuire (2007). The two columns are oriented for minor-axis bending. The nominal yield stress and modulus of elasticity for the members are 345 MPa and 200 GPa, respectively. The frame has an initial out-of-plumbness of 1/500 towards the right. The reference load $P_0 =$ 484.86kN.

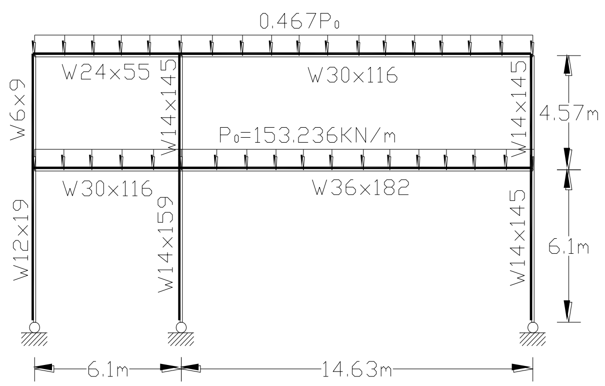


Figure 2 Example 2: 2-bay, 2-storey unsymmetrical frame.

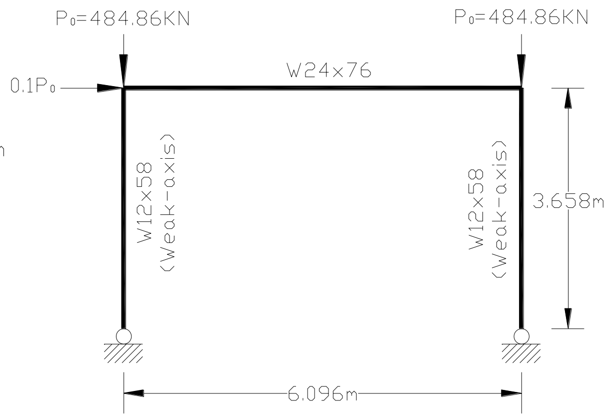


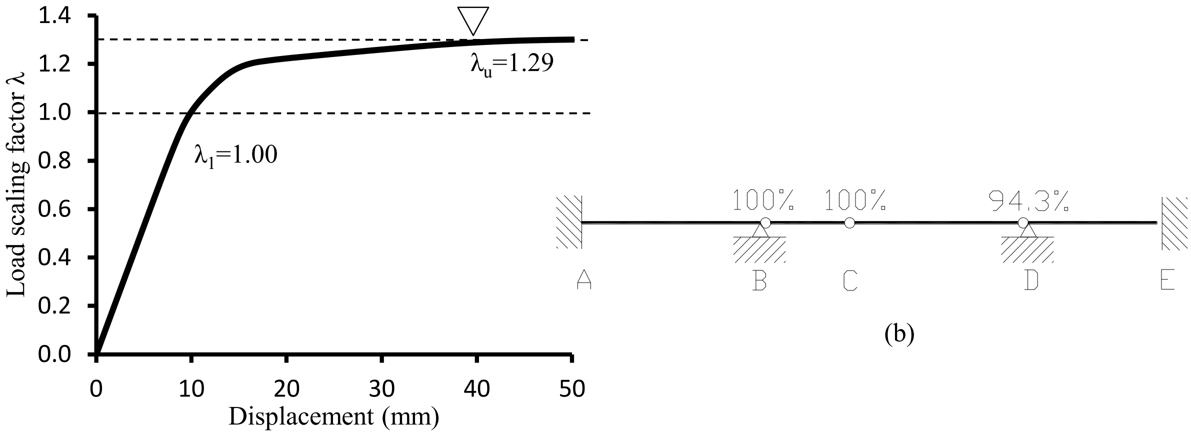
Figure 3 Example 3: slender portal frame.

In all examples, all beams and columns are compact and laterally braced so that the plastic capacity of each section can be achieved without local buckling. Connections are assumed to be fully rigid; compliance of the connections is not considered.

Second-order elastic/inelastic analyses were performed using the software OpenSEES. Initial geometric imperfections were modeled directly in the analysis by offsetting the relevant nodes from their nominal positions. For the inelastic analysis, displacement-based, fibre-type beam elements were used to model the spread of plasticity throughout the cross-sections and along the members. The stress–strain curve is assumed to be elastic-perfectly plastic. Residual stresses are assumed to distribute according to the pattern suggested in (Galambos and Ketter, 1961), i.e., linearly varying across the flanges and constant tension in the web. In the analyses, the loads applied on the structures are incrementally increased; a load scaling factor λ is used to (proportionally) increase the reference loads P_0.

Failure behaviour: Example 1

Inelastic analysis shows that significant yielding occurred first at Point C and that the second plastic hinge developed at Point B. Eventually, the third plastic hinge appeared at Point D; at which point, the continuous beam reached its load-carrying limit. Figure 4(a) plots the load scaling factor versus the vertical displacement at Point C when the member yield stress and modulus of elasticity are at 0.9 times their nominal values. The first plastic hinge developed at $\lambda = 1.0$. It can be seen that the ultimate load factor (λ_u) is 1.29. Figure 4(b) shows the sections with significant yield ratio (percentage of yielded cross-sectional area) at the ultimate limit point λ=1.29.



(a)

Figure 4 Example 1: (a) load-displacement at Point C, and (b) section yield ratios at λ_u = 1.29.

Table 1 compares the strength of the continuous beam determined by four different methods: 1) DAM, 2) (traditional) plastic design method, 3) inelastic method in AIS360-10, and 4) inelastic method in AS4100. The traditional plastic design of continuous beams is based on the plastic mechanism loads, and a factor of 0.9 is applied to the plastic moment capacities of all members. Table 1 shows that the ultimate load factors from the plastic design method and the inelastic method in AISC360 are identical, i.e., λ_u =1.29, which is 29% higher than the DAM. This suggests that there is 29% remaining strength to sustain further loading after first yielding. This example also shows that for this simple continuous beam, the traditional plastic analysis method and the inelastic method in AISC360 give the same results. This is to be expected. On the other hand, the λ_u given by the inelastic method of AS4100 is 0.99, which is essentially the same as the DAM, and is significantly conservative compared to the inelastic method of AISC360. As AS4100 requires a section yield capacity check, the system ultimate limit point is reached at the first plastic hinge. This negates any post-yield strength that could be gained by using inelastic analysis for a system-based design.

Table 1 Ultimate load factor (λ_u) from different design methods, Example 1.

Design Method	λ_u
DAM	1.00
Plastic design	1.30
Inelastic (AISC360)	1.30
Inelastic (AS4100)	0.99

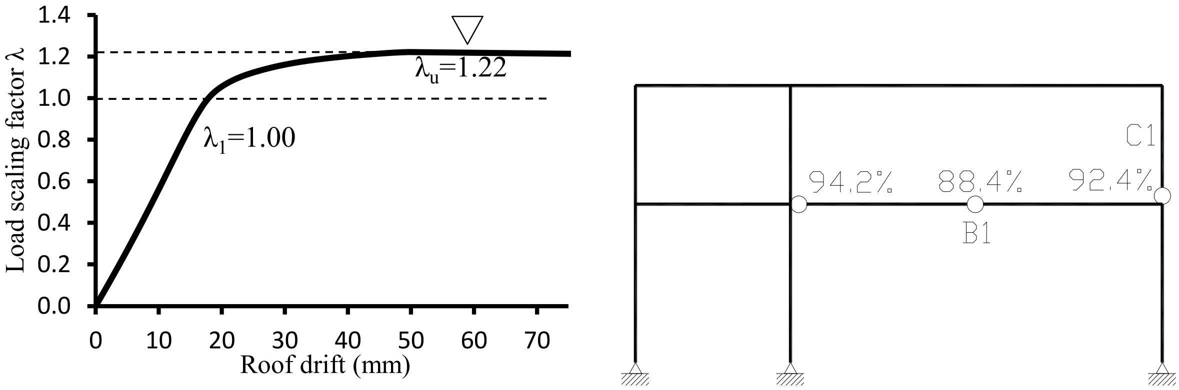
Table 2 Ultimate load factor (λ_u) from different design methods, Example 2.

Design Method	λ_u
DAM	1.00
Inelastic (AISC360)	1.22
Inelastic (AS4100)	1.00

Failure behaviour: Example 2

For the second example, the inelastic method of AISC360 indicates that beam B1 developed two plastic hinges, firstly at the left end of B1, then in the middle. The load scaling factor associated with the occurrence of the first plastic hinge is λ=1.0. Further increasing the load to λ=1.22, the third plastic hinge occurred at the bottom end of column C1. At this point, the system reached its load carrying capacity and is in a state of incipient instability. Figure 5 shows the load-roof drift response and the section yield ratios at the limit point for Example 2. The analysis is based on values of yield stress and elastic modulus of 0.9 times their nominal values.

Table 2 presents the system strengths given by three different methods: 1) DAM, 2) inelastic design method in AIS360-10, and 3) inelastic design method in AS4100. Similar observations were made as in Example 1. The inelastic method of AS4100 and the DAM gave the same results of λ_u = 1.0, which corresponds to the load when the first plastic hinge was developed. The "true" ultimate strength given by the system-based inelastic method-AISC360 is 1.22, which is 22% greater than that based on the first hinge approach.



(a) (b)

Figure 5 Example 2: (a) load-roof drift behaviour, and (b) section yield ratios at λ_u = 1.22.

Failure behaviour: Example 3

The slender portal frame fails essentially elastically. Figure 6 plots the load-drift response from the inelastic method of AISC360-10 based on values of yield stress and elastic modulus of 0.9 times their nominal values. At the ultimate point, the top ends of the two columns are partially yielded, with a yield ratio of about 55%. Other parts of the structure are in the elastic range. Table 3 summarizes the ultimate load factors from the DAM, the inelastic method in AISC360 and the inelastic method in AS4100. It can be seen that the DAM and the inelastic method in AISC360 predict the same system strength. This is not surprising since the structure behaves largely in its elastic range when the slender columns buckle elastically. From Table 3, it can be seen that the inelastic method in AS4100 over-predicted the system strength by 4%. The inelastic method in AS4100 stipulates a section yield capacity check with the section axial and moment capacities reduced by a resistance factor of 0.9. This practice is equivalent to performing inelastic frame analysis with a reduced yield stress $0.9F_y$ and nominal values of stiffness for all members. Since the structure fails due to elastic buckling of the slender columns, reducing F_y only will not have correct effect on this slender structure. In this example, using a reduced yield surface does not sufficiently factor down the nominal system strength. In comparison, the inelastic method in AISC360 reduces both the strength and stiffness of all members by a factor of 0.9. The 0.9 factor on stiffness ensures that the strength of a frame with slender members failing elastically will also be factored downward.

Table 3 Ultimate load factor (λ) with different analysis methods in example 3.

Design Method	λ_u
DAM	1.00
Inelastic (AISC360)	1.00
Inelastic (AS4100)	1.04

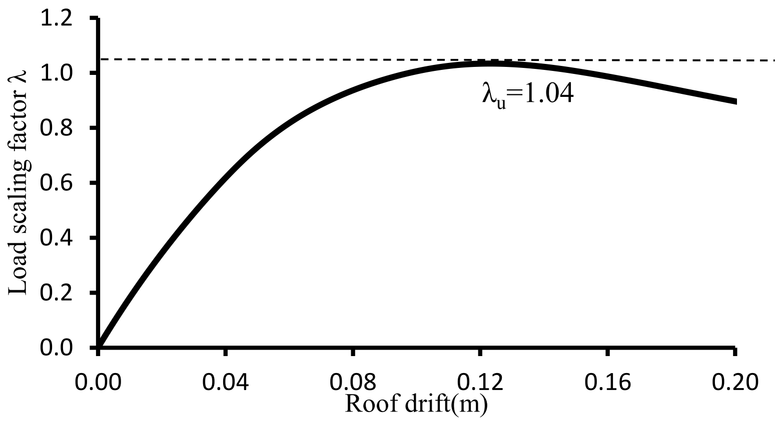


Figure 6 Example 3: load-roof drift behaviour at $\lambda_u = 1.04$.

DISCUSSIONS AND CONCLUSIONS

This paper presents a comparison of the design strengths of three steel structures using the DAM, the inelastic method in AISC360, and the inelastic method in AS4100. The three structures exhibit different ductility (capacity of load redistribution) and failure modes.

The continuous beam and the two-bay two-storey frame have significant capacities of load redistribution after first yielding. Using the inelastic method in AISC360 could gain significant post-yield strength; the system strengths are 20-30% larger than those from the DAM. In the continuous beam example, the result from the inelastic method in AISC360 agrees with that from the traditional plastic design method for continuous beams which is based on the plastic mechanism loads. For these two examples, the inelastic method in AS4100 gives the same system strength as the DAM. This is because AS4100 requires a section yield capacity check. As a result, the system ultimate limit point is reached at the first plastic hinge. This negates any post-yield strength that could be gained by using inelastic analysis for a system-based design. If the inelastic analysis is only used to obtain member forces for use in member/section capacity checks, the main benefits of using inelastic analysis cannot be fulfilled.

Because the slender portal frame fails essentially elastically, the inelastic method in AISC360-10 and the DAM give the same system strength. In this example, the inelastic design method does not lead to more economic design (compared to the member-based DAM). On the other hand, the inelastic method of AS4100 over-predicted the system strength. This is because AS4100 only reduces the strength of all members, which has no effect on slender structures failing elastically.

The inelastic design provisions in AS4100 appear to be problematic, for the two reasons summarized above, whereas the inelastic design method of AISC360-10 is more rational. It represents a system-based design methodology, and may give more economic design than the traditional member-based first plastic hinge design approach. The gain of post-yield strength by using system-based inelastic design depends on the nature (ductility and redundancy) of the system.

ACKNOWLEDGMENTS

This research is supported by Australian Research Council under Discovery Project Grant DP110104263. This support is gratefully acknowledged.

REFERENCES

AISC360-10 (2010), “Specification for Structural Steel Buildings”, *American Institute of Steel Construction (AISC),* Chicago, Illinois.

AS4100 (1998), *Australian Steel Structures Standard*, Sydney, NSW 2001, Australia.

Chandrangsu, T & Rasmussen, KJR. (2011). "Investigation of geometric imperfections and joint stiffness of support scaffold systems", *Journal of Constructional Steel Research*, 67(4), 576-584.

Chen, WF & Kim, SE. (1997). "LRFD steel design using advanced analysis", *CRC press*, Vol. 13.

Clarke, MJ, Bridge, RQ, Hancock, GJ, & Trahair, NS. (1992). "Advanced analysis of steel building frames”, *Journal of Constructional Steel Research*, 23(1), 1-29.

Galambos, TV & Ketter RL. (1961), "Columns under combined bending and thrust", *Transactions of the American Society of Civil Engineers*, 126(1), 1-23.
Hwa, K. (2003). "Toward advanced analysis in steel frame design", PhD. Dissertation, *University of Hawaii at Manoa*.
Kim, SE & Chen, WF. (1996). "Practical advanced analysis for steel frame design", *In Analysis and Computation* , ASCE, 19-30.
King, WS, White, DW, & Chen, WF. (1992). "Second-order inelastic analysis methods for steel-frame design", *Journal of Structural Engineering*, ASCE. 118(2), 408-428.
Liew, JYR, & Chen, WF. (1991). "Refining the plastic hinge concept for advanced analysis/design of steel frames", *Journal of Singapore Structural Steel Society*, Steel Structures, Vol, 1(2).
Liew, JYR. (1992), "Advanced analysis for frames design", PhD. Dissertation, School of Civil Engineering, *Purdue University*, West Lafayette.
Liew, JYR, White, DW, & Chen, WF. (1993). "Second-order refined plastic-hinge analysis for frame design. Part I", *Journal of Structural Engineering*, ASCE. 119(11), 3196-3216.
Shayan, S, Zhang, H & Rasmussen, KJR (2013), "System-based design provisions for 2D steel frames by advanced analysis", Research Report, School of Civil Engineering, *the University of Sydney*.
Zhang, H, & Rasmussen, KJR (2013). "System-based design for steel scaffold structures using advanced analysis", *Journal of Constructional Steel Research*, 89, 1-8.
Ziemian, RD, & McGuire, W. (2007). "Learning Module Number 6: Beam Design by Elastic and Inelastic Analyses", *Tutorials for MASTAN2: Version3*. Retrieved from http://www.mastan2.com/stabilityfun.html
Ziemian, RD, & McGuire, W. (2007). "Learning Module 9: Design by the Direct Analysis Method", *Tutorials for MASTAN2: Version3*. Retrieved from http://www.mastan2.com/stabilityfun.html
Ziemian, RD, McGuire, W & Deierlein, GG. (1992). "Inelastic limit states design. Part I: Planar frame studies", *Journal of Structural Engineering*, 118(9), 2532-2549.