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Analysis and design for stability in the U.S. - An overview

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Abstract. This paper describes the theoretical background and underlying principles behind the American Institute of Steel Construction Load and Resistance Factor Design (AISC LRFD) Specification for the analysis and stability design of steel frames. Various analysis procedures that can take into consideration the effects of member instability, frame instability, member-frame interaction, geometric imperfections, and inelasticity are reviewed. Design approaches by which these factors can be incorporated in the design of steel moment frames are addressed. Current specification guidelines for member and frame design in the U.S. are summarized. Examples are given to illustrate the validity of the design equations. Some future directions for the analysis and stability design of steel frames are discussed.

Key words: stability; nonlinear effects; advanced analysis; limit states design; steel frames.

1. Introduction

Because of the inherent strength and stiffness of steel, structural steel members are usually quite slender when compared to reinforced concrete members for a given design strength. Consequently, geometrical nonlinearity or nonlinearity due to change in geometry of the structure becomes an important design consideration. Furthermore, because of world-wide acceptance of the limit states design philosophy in which structures and structural components are designed according to their limits of usefulness, material nonlinearity in the form of yielding or inelasticity will also be an important design issue.

In addition to geometrical and material nonlinearities, other contributing factors to nonlinearity in steel frame structures include material and member/frame imperfection effect, bowing effect, cross-section warping effect, local instability effect, local-global instability interaction effect, finite 3-*D* rotational effect, strain rate effect, work-hardening effect, axial force-moment-shear-torsion plastic interaction effect, and cyclic plasticity effect, etc. Of these various effects, material imperfection in the form of residual stresses, member imperfection in the form of out-of-straightness, frame imperfections in the form of story out-of-plumbness, overall frame non-verticality, and member-frame interaction are important factors that need to be considered in assessing overall frame response. The other effects, though important in some situations, are usually minor in comparison and so they can ordinarily be ignored.

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In the following section the primary factors that contribute to nonlinearity in steel frame structures - namely, geometrical and material nonlinearities as well as material, member and frame imperfections - will be discussed. Analysis methods by which these effects can be accounted for in design will be reviewed. This is followed by a discussion of the AISC LRFD approach for incorporating these nonlinear effects in design. An alternative approach referred to as the notional load approach by which these effects can be incorporated directly in structural analysis will be presented. Examples will be shown to demonstrate how the code equations can be used to capture these nonlinear effects. The paper will conclude with a discussion of what direction stability design might take in the future.

2. Nonlinear behavior of steel structures

As mentioned in the preceding section, nonlinearity in steel frame structures is attributed to a number of factors. Of the various elements that affect frame response to applied loads, geometrical and material nonlinearities are perhaps two of the most important factors. Geometrical nonlinearity arises when a change in geometry of the structure or structural component changes the response characteristics of the structure. Geometrical nonlinearity may be the result of member instability (P-δ effect) and/or frame instability (P-Δ effect). P-δ effect arises when the axial force in a member acts through the curvature of the member, and P-Δ effect arises when the axial force in a member acts through the relative end displacements of the member. These two P-Delta effects are shown schematically in Fig. 1. P-δ effect is present as long as the member experiences flexural deformation and P-Δ effect is present as long as the member undergoes sway movement. Under a compressive force, both the P-δ and P-Δ effects tend to aggravate the deflection and increase the moment in the member, and so they must be accounted for in design. Some analytical and design approaches by which these effects can be incorporated into analysis

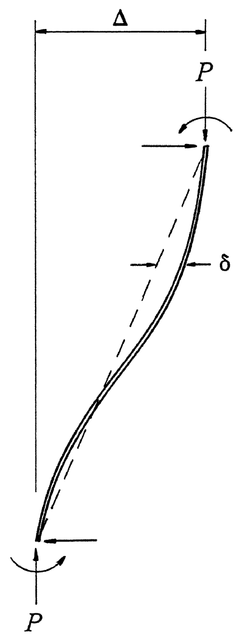


Fig. 1 Member (P-δ) and frame (P-Δ) instability effects

and design of steel frame structures will be discussed in subsequent sections. It suffices to say at this point that neglecting geometrical nonlinearity often leads to an unconservative design.

Material nonlinearity arises as a result of the material not obeying Hooke's Law (i.e., the stress-strain relationship of the material is nonlinear) or the material becoming inelastic (i.e., the stress in the member exceeds the yield strength of the material). For structural steel, even though the stress-strain relationship is mostly linear in the elastic region, nonlinear behavior may occur well below the yield strength of the material because of the presence of residual stresses. Residual stresses are self-equilibrating stresses present in hot-rolled and welded sections as a result of uneven cooling rate in different parts of the cross-sections. Residual stresses cause early yielding in some fibers of the cross-section and thus contribute to the nonlinear behavior of the cross-section and the member. Residual stresses affect both members under compression and members under flexure.

The effect of residual stresses on cross-sections under compression is depicted in Fig. 2 in which the stress-strain behavior of a coupon test and a stub column test are shown. Although the coupon, which is free of residual stresses, exhibits an elastic-perfectly plastic stress-strain behavior, the stub column, which has residual stresses, exhibit a nonlinear stress-strain behavior at an average stress well below the yield strength of the material. This early yielding is attributed to the compressive residual stresses that are present in the cross-section of the stub column. For cross-sections under flexure, residual stresses cause the sections to yield at a moment below the yield moment of the cross-sections. The residual stress effect on cross-sectional strength is depicted in Fig. 3 in which the moment-curvature-thrust (M-Φ-P) curves of a typical I-section are shown. Although residual stresses have no effect on the plastic moment strength M_p of the cross-section, they do cause early yielding and reduce the moment capacity of the cross-section for moments below M_p. Regardless of the presence of residual stresses, the nonlinear behavior of the M-Φ-P relationship is due to a phenomenon called cross-section plastification. Under flexure, fibers in a cross-section are not uniformly strained; fibers that are further away from the neutral axis will strain more than fibers that are nearer to the neutral axis. As a result of this uneven straining, yielding is a progressive process commencing at the extreme fibers and spreading towards the inner fibers. The transition from the yield moment M_y to M_p is therefore a gradual (nonlinear) process.

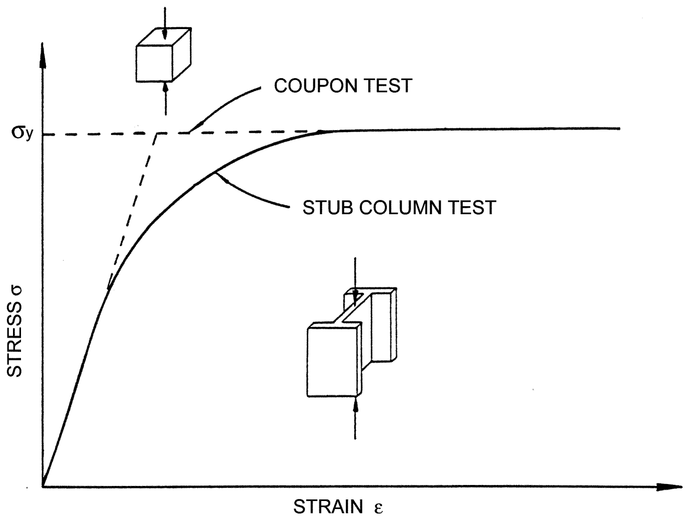


Fig. 2 Coupon and stub column stress-strain curves

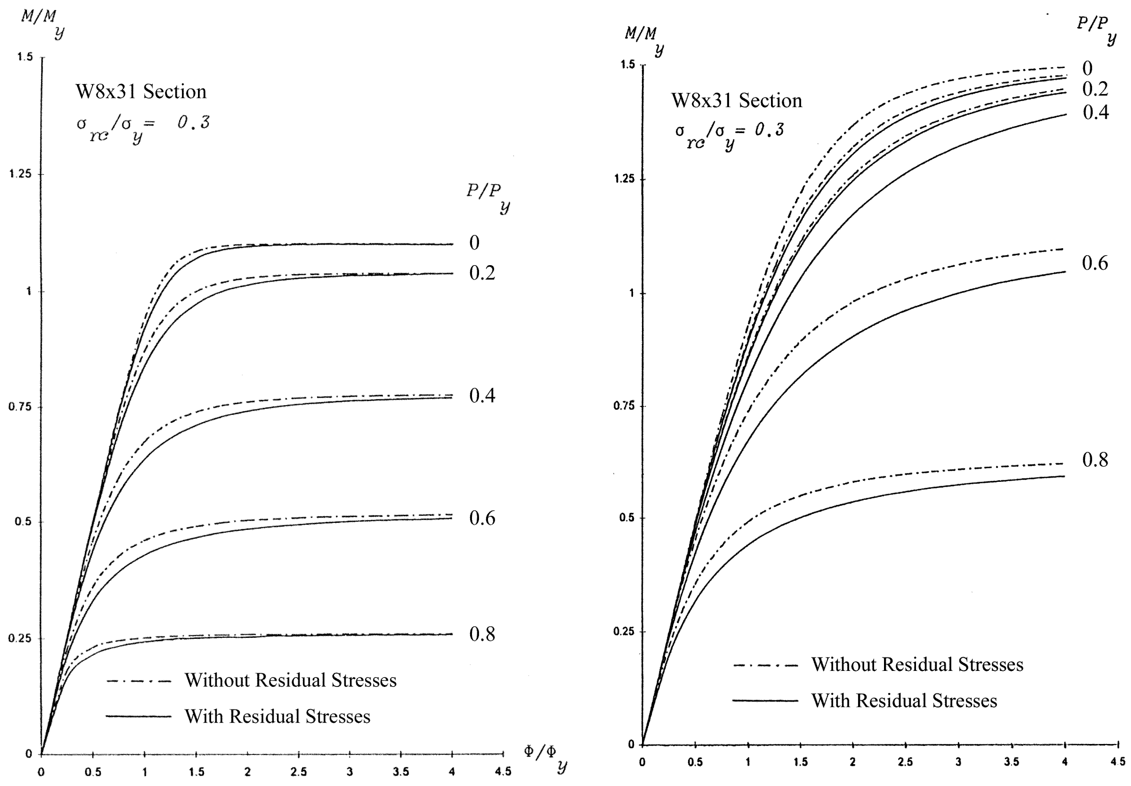


Fig. 3 Cross-section moment-curvature-thrust (M-Φ-P) relationship

The presence of residual stresses further complicates this progressive yielding process because fibers that are equal distance from the neutral axis may yield at different stages depending on whether the residual stresses in those fibers are compressive or tensile. The process becomes more complex if in addition to bending moment, the cross-section is subjected to other internal stress resultants such as axial/shear forces, and torsion. The simultaneous occurrence of these internal forces and moments creates a very complex pattern of stress distribution over the cross-section, making the development of constitutive relationships for these stress resultants very challenging.

It should be noted that residual stresses not only affect cross-section behavior, they also influence member behavior. Very often, the moment in a member is not constant along its length. If a moment gradient exists in the member, fibers at different locations along the member length will yield at different stages. As the applied loads increase, the spread of yield occurs not just within a cross-section, but along the member length. Residual stresses complicate this progressive yielding process and their effect on overall member response is depicted schematically in Figs. 4 and 5. In these figures the effect of residual stresses on the compressive strengths of columns and lateral torsional buckling strength of beams are shown. Residual stresses tend to reduce the compressive strengths of columns and the lateral torsional buckling strengths of beams. In design, this instability related strength reduction is often accounted for implicitly in the member design equations and rigorous analysis by which residual stresses are explicitly accounted for is seldom performed.

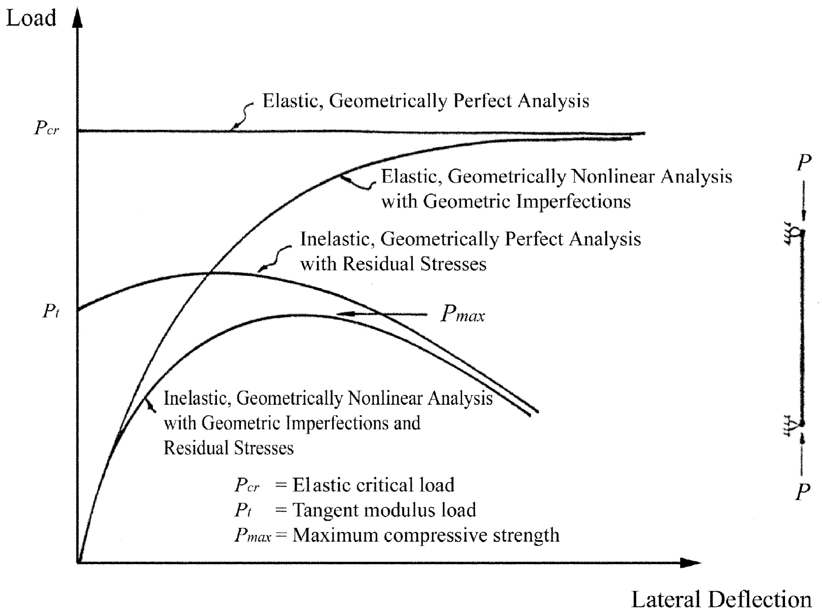


Fig. 4 Compression member load deflection curves

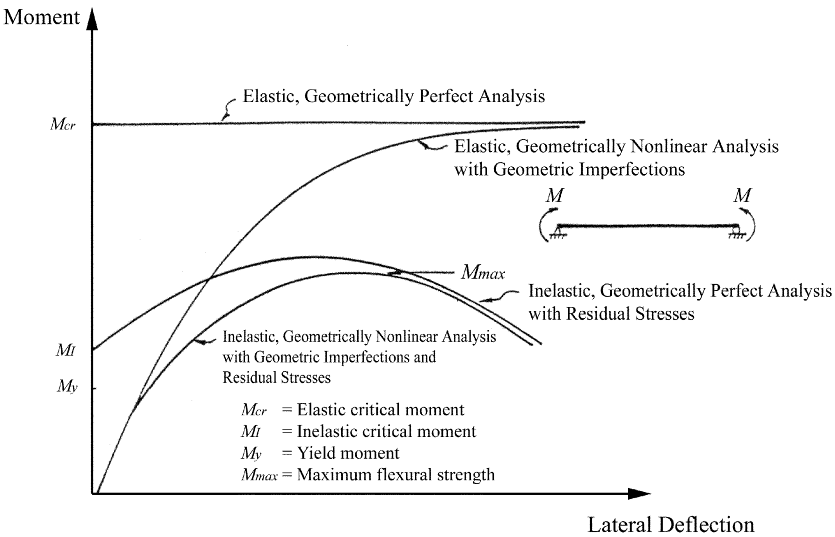


Fig. 5 Flexural member load deflection curves

In general, geometric imperfections can be classified into: (1) member out-of-straightness; (2) story out-of-plumbness; and (3) global frame non-verticality. These various forms of imperfections are shown schematically in Fig. 6. According to the AISC Code of Standard Practice for Steel Buildings and Bridges (Code 2000), the maximum fabrication or erection tolerances for (1) member out-of-straightness is δ_o/L=0.001, where L is the length of the member between laterally supported points; (2) story out-of-plumbness is Δ_o/h = 0.002, where h is the story height; and (3) global frame non-verticality is: For exterior columns, 1 in. (25 mm) toward or 2 in. (50 mm) away from the building line in the first

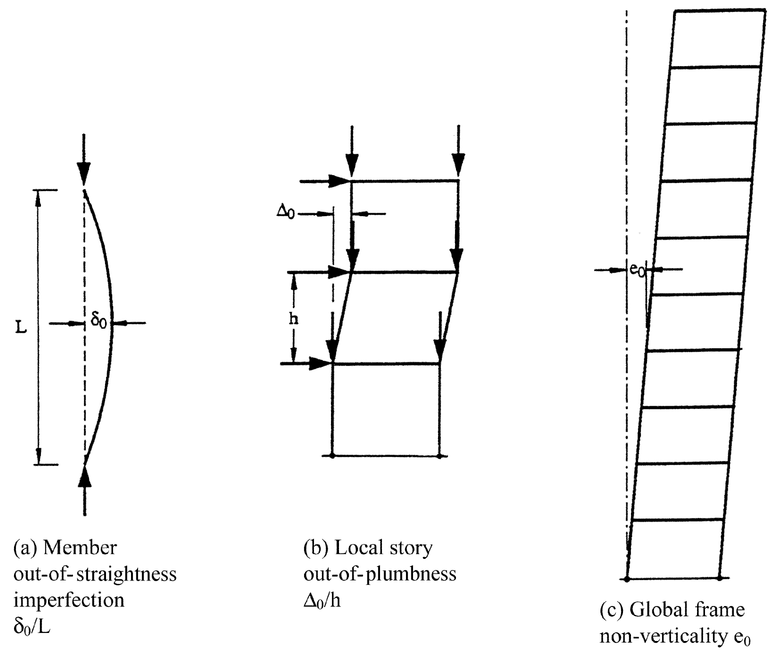


Fig. 6. Geometric imperfections

20 stories; above this level, the deviation can be increased by 1/16 in. (1.6 mm) for each additional story up to a maximum of 2 in. (50 mm) toward or 3 in. (75 mm) away from the building line. For columns adjacent to elevator shafts, 1 in. (25 mm) from established column line in the first 20 stories; above this level, the deviation can be increased by 1/32 in. (0.8 mm) for each additional story up to a maximum of 2 in. (50 mm).

From an analysis standpoint, an exact modeling of geometric imperfections is not an easy task. The difficulty arises because of the randomness in distribution of the magnitudes, shapes and orientations of imperfections throughout the frame. Under certain combinations of loading conditions and imperfection distributions, geometrical imperfections can actually be beneficial to the strength and stability of the frame. For instance, if lateral loads are applying in a direction opposite to the story out-of-plumbness and global non-verticality of a frame, the P-Δ moments so generated by these imperfections can actually reduce the primary moments generated by the lateral loads. In reality, the probability that all story out-of-plumbness and frame non-verticality will occur in the same direction is extremely low. As a result, recourse to statistical method to determine this random distribution of imperfections is indispensable if a realistic model capable of incorporating these imperfections explicitly in a rigorous analysis is to be used. For design purpose, structural imperfections are often accounted for implicitly in the design equations through calibrations against experimental data or analytical results that include such imperfections and by the use of safety factors (in the Allowable Stress Design format) or resistance factors (in the Load and Resistance Factor Design format).

In the U.S., the out-of-straightness effect in columns is accounted for implicitly in the LRFD column strength equations. The LRFD column strength equations were developed based on curve-fitting of data generated numerically for the compressive strengths of geometrically imperfect columns with residual stresses. While the out-of-plumbness effect is not explicitly accounted for in the column equations, the use of resistance factors implicitly allows for any deleterious effect out-of-plumbness may have on column strength. The implicit consideration of geometrical and material imperfections in the member design equations coupled with the use of resistance factors greatly simplify the design process.

In Australia, Canada and Europe, the effects of member out-of-straightness and residual stresses are accounted for implicitly in the column strength equations in a manner similar to the U.S. The out-of-plumbness effect is accounted for explicitly in the analysis by the use of notional lateral loads. Notional loads are fictitious loads (expressed as a fractional multiple of the story gravity loads) to be applied (in conjunction with the real lateral loads) to the frame whose purpose is to generate secondary moments induced by the out-of-plumbness effect. The U.S. has incorporated this notional load concept in its draft specification (LRFD 2003). The concept of notional lateral loads will be explored in more detail in a later section. In the following section, a compendium of analytical and design approaches by which a designer can incorporate the aforementioned geometrical and material nonlinearities in assessing the required strengths of steel frame structures will be given. The advantages and drawbacks of each approach will also be discussed.

3. Geometrical and material nonlinear analyses

For sake of clarity, we shall first address the problem of geometrical nonlinearity in steel frame analysis. The discussion of material nonlinearity and means by which it can be incorporated into the analysis will be deferred. By ignoring material nonlinearity at the outset, we are in effect limiting the problem to one of elastic. Thus, the usual assumptions for the analysis of elastic structures such as: (1) Material is homogeneous and isotropic; (2) Stress-Strain behavior obeys Hooke's Law; and (3) Plane sections before bending remain plane after bending, etc., will hold.

3.1. Geometrical nonlinear analysis

Depending on the level of sophistication and degree of accuracy one wants, geometrical nonlinear analysis can be formulated and carried out in a number of ways. This section gives a succinct discussion of the various approaches by which one can account for geometrical nonlinearity in the analysis.

3.1.1. Rigorous approaches for geometrical nonlinear analysis

Generally speaking, geometrical nonlinear steel frame analysis can be formulated using either a beam-column approach or a finite element approach. In the beam-column approach, the P-δ and P-Δ effects are accounted for by the use of stability stiffness functions (see, for example, Chen and Lui 1987, 1991) in the stiffness formulation of the force-displacement relationship of the member. In the finite element approach, these P-Δ effects are accounted for by the use of a geometrical stiffness matrix in the element stiffness formulation (Gallagher and Padlog 1963, McGuire *et al.* 2000). These two approaches for geometrical nonlinear frame analysis are well documented in the literature and so detailed descriptions will not be given here. It suffices to say that since the stability stiffness functions and the geometrical stiffness matrix are functions of the axial force in the member, which is not known in advance, iterative algorithm must be used to obtain solution. Very often, the axial force calculated in a previous cycle of iteration is used to formulate the stiffness matrix for the present cycle of iteration. Convergence is said to have achieved when the change in displacement and/or force results become negligible.

One major difference between the beam-column approach and the finite element approach is the way the structure needs to be modeled. For the same degree of accuracy, the number of elements required to model the structure is usually higher for the finite element approach. This is because the stiffness matrix formulated based on finite element is less accurate than that formulated using stability functions. In fact, it can be shown readily that the first-order and geometrical stiffness matrices used in finite element analysis can be obtained from the first and second term of a Taylor series expansion of the stability functions, respectively. However, with the availability of powerful desktop computers with ever-increasing speed and memories, coupled with the use of enhanced graphical software capable of automating data input and output, the inconvenience of having to use more elements to model a structure is diminishing. Furthermore, finite element has an advantage over the beam-column formulation in that extension to 3-D analysis including the effect of warping can be more readily achieved (Hsieh *et al.* 1989).

A method that retains some of the accuracies of the beam-column approach but avoids the use of multiple elements to model a member was introduced by Chan and Zhou (1994). The element used in this approach is referred to as the point-wise equilibrium polynomial (PEP) element. The approach requires the use of a special element stiffness matrix derived using finite element technique. However, unlike the regular finite element in which a Hermitian (third-order) polynomial is used to describe the transverse displacement of the element, a fifth-order polynomial is used in the PEP element. Because a fifth-order rather than a third-order polynomial is used, two additional conditions need to be enforced in the derivation of the element stiffness matrix. The two additional conditions provided are the shear and moment equilibrium conditions at midspan of the element. Results provided by Chan and Zhou (1994) have shown that reasonable accuracies can be achieved by using just one element per member in the structure model.

3.1.2. Simplified approaches for geometrical nonlinear analysis

All three approaches described in the foregoing, namely, the beam-column approach, the finite element approach, the PEP approach, require the use of special element stiffness matrices to model geometrical nonlinearity. As a result, approach specific computer programs are needed to perform the geometrical nonlinear analysis. In this section several approaches that do not require the use of special element matrices and can be carried out using any readily available first-order computer programs are discussed. One such approach is the pseudo load approach (Lui 1988). In this approach, the P-δ and P-Δ effects are accounted for explicitly by subjecting the member to pseudo in-span transverse load and pseudo end shears, respectively, as shown in Fig. 7. The pseudo in-span transverse load is obtained from the equation PM/EI where P is the axial force in the member, M is the first-order moment distribution along the member, and EI is the flexural rigidity of the member. This load is to be applied in a direction consistent with the sense of M; i.e., if the moment causes tension on one side of the member, the pseudo in-span transverse load is to be applied in a direction so as to increase the tensile stress on that side of the member. The pseudo end shears at the A and B ends of a member are obtained from the equation $P(dy/dx)_A$ and $P(dy/dx)_B$, where P is the axial force in the member, $(dy/dx)_A$ and $(dy/dx)_B$ are the end slopes at the A and B ends of the member, respectively. These pseudo end shears are to be applied in directions so as to simulate the rotational effect impart to the member by the P-Δ effect. When more than one member meet at a joint, the pseudo end shears are to be added algebraically at the joint to form a joint load. Note that the pseudo in-span load and the pseudo end shears constitute an equilibrium force set on the member. To carry out the analysis, the structure is first analyzed using any available first-order analysis technique. Based on the results of this analysis, pseudo in-span transverse loads and member end shears are calculated for all members of the frame. A reanalysis is performed with the structure subject to both the ‘real’ and the pseudo loads. The procedure is repeated until the solution converges. For ordinary frameworks, convergence can be easily achieved in just two or three

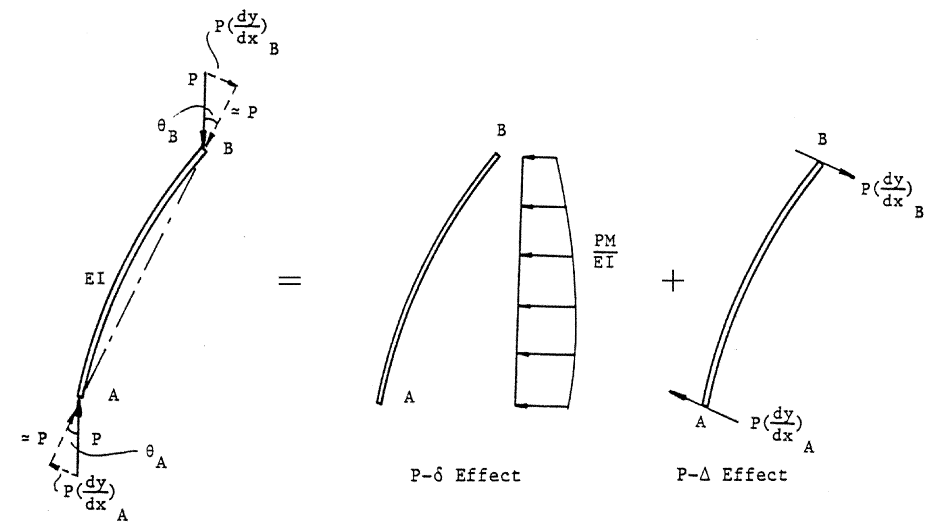


Fig. 7 Pseudo load method

cycles of analyses. The physical significance of the pseudo load method can be explained in terms of a power series approximation in that each cycle of analysis generates a term in the Taylor Series expansion of the ‘exact’ elastic beam-column solution. The method has also been extended to inelastic analysis (Lui and Zhang 1990) and inelastic analysis with geometric imperfections (Lui 1992).

While the pseudo load method can account for both the P-δ and P-Δ effects, the P-Δ effect is usually more pronounced for ordinary unbraced steel moment frames. For a given member with length L and flexural rigidity EI, neglecting the P-Δ effect will introduce a maximum error of only 5% in the member bending stiffness even for the most severe loading condition when $PL^2/EI \le 0.56$. Simplified approaches by which only the frame instability (i.e., P-Δ) effect is accounted for have been proposed. These include the Nixon’s negative brace method (Nixon *et al.* 1975), Rutenberg’s fictitious column method (Rutenberg 1981), Adam’s P-Δ iterative method (Adams 1974) and Stafford Smith’s gravity load iterative method (Stafford Smith and Gaiotti 1988). Details of these methods are described in Chen and Lui (1991) and so only the essence of the methods will be outlined here.

Both the Nixon’s negative brace method and the Rutenberg’s fictitious column method are non-iterative methods that make use of the notion that the presence of P-Δ effect tends to reduce the stiffness of the frame. This reduction in frame stiffness is modeled by the introduction of fictitious braces having negative axial stiffness in the Nixon’s method, and by the introduction of fictitious columns having negative shear or flexural stiffness in the Rutenberg’s method. In using either the Nixon’s or the Rutenberg’s method, an analyst only needs to add the appropriate fictitious elements to the structural model and carry out a first-order analysis. No iteration is required and the results so obtained will automatically include the destabilizing effect due to frame instability. The major disadvantage of these two methods is their inability to account for member instability (i.e., P-δ) effect. Member instability effect is usually not as significant as frame instability effect except when the axial force in the member is extremely high as for columns in the lower stories of high-rise buildings or when the member is subject to single curvature bending as for columns in braced frames or when the member is unduly slender. In these situations, care must be exercised in applying the Nixon’s or the Rutenberg’s method for geometrical nonlinear analysis.

In the P-Δ iterative method and the gravity load iterative method, frame instability is accounted for by subjecting the frame to fictitious loads. In the P-Δ iterative method, the P-Δ effect is simulated by subjecting the frame to a set of fictitious lateral loads given by

$$H_i = \frac{\sum P_i \Delta_i}{h_i} - \frac{\sum P_{i+1} \Delta_{i+1}}{h_{i+1}} \tag{1}$$

where H_i is the fictitious lateral load to be applied at story i, ΣP_i and ΣP_{i+1} are the sum of column axial forces at stories i and $i+1$, Δ_i and Δ_{i+1} are the interstory deflections of stories i and $i+1$, h_i and h_{i+1} are the heights of stories of i and $i+1$, respectively.

To implement the P-Δ iterative method of geometrical nonlinear frame analysis, the frame is first analyzed using any first-order analysis technique. Using the interstory deflections obtained from this analysis, fictitious lateral loads are calculated from Eq. (1). A second analysis is then performed with the frame subject to both the real and the fictitious lateral loads. The process is repeated until the interstory deflections calculated in two consecutive cycles of analyses do not change appreciably.

In the gravity load iterative method, the P-Δ effect is accounted for by applying gravity loads on a fictitious bay composed of axially rigid columns with zero flexural rigidities. This fictitious bay is appended to the frame by rigid links. To implement the method, a lateral load analysis is first performed on the frame. The horizontal deflections obtained from this lateral load analysis are used to define the joint coordinates of the fictitious bay. Cumulative gravity loads (i.e., ΣP) are then applied to the columns of the fictitious bay. The additional horizontal deflections so obtained from this gravity load analysis are used to define the new joint coordinates of the fictitious bay. Another gravity load analysis is performed on the new geometry of the fictitious bay. The process is repeated until the additional horizontal deflections obtained in the analysis become negligible.

The P-Δ iterative method and the gravity load iterative method will produce identical results if implemented properly. The only difference is that the gravity load iterative method tends to converge more rapidly. Nevertheless, it should be mentioned that neither of these methods can account for the P-δ effect and so the validity of these methods may be questionable if member instability effect is important. In addition, since the effect of column gravity loads is lumped in the term ΣP, the P-Δ moments so calculated may not be accurate if the axial forces in the columns vary significantly across the width of the frame. This happens when the frame has multiple bays with large spans.

3.2. Geometric Imperfections

Geometric imperfections in the form of member out-of-straightness, story out-of plumbness and global non-verticality as shown schematically in Fig. 6 are always present in real frameworks. While the effect of member out-of-straightness can be conveniently accounted for implicitly in the column equations, the effect of story out-of-plumbness and the resulting global non-verticality must be accounted for in some other manners. At present, three approaches have been proposed to account for story out-of-plumbness effect: (1) explicit modeling, (2) notional lateral load, and (3) reduced modulus. In the explicit modeling approach, story out-of-plumbness is explicitly modeled in the structural model. To reflect the presence of out-of-plumbness and frame non-verticality, joint coordinates for the columns are defined so that they do not lie on a plumb line. The difficulty that arises with this type of modeling is that the distribution of story out-of-plumbness is a random process, and so an 'exact' model of the frame is very difficult, if not impossible, to create. Recognizing the random nature of the problem and the fact that geometric imperfections for multistory multibay frames should be reduced because it is highly unlikely that all columns will lean in the same direction, Eurocode 3 (CEN 1992) recommends that the magnitude of story out-of-plumbness be reduced by two factors (k_3 and k_4) to account for the beneficial effect that arises when the number of columns per story and the number of stories per frame increase.

In the notional lateral load approach (to be described in a later section), the effect of story out-of-plumbness is accounted for by subjecting the frame to a set of notional lateral loads whose magnitudes are expressed as a fraction of the gravity loads acting on the frame. The notional lateral load approach is the recommended approach in the Australian, Canadian, and European steel design codes, and is contained in the draft U.S. steel design specification.

In the reduced modulus approach (Kim and Chen 1996a, 1996b), a reduction factor is applied to the flexural rigidity of the member to account for effect of geometric imperfections. The rationale behind this approach is that geometric imperfections tend to reduce the stiffness of the member. This stiffness reduction can be readily effected by reducing its flexural rigidity. Examples given by Kim and Chen (1996a, 1996b) have demonstrated the validity of this approach in modeling geometric imperfections in both braced and unbraced steel frame structures.

It should be noted in passing that in addition to accounting for the out-of-plumbness effect, both the notional load and the reduced modulus methods can be extended to account for material nonlinear effect in the form of yielding. This is usually done through calibration with known inelastic results.

3.3. Material nonlinear analysis

As discussed earlier, material nonlinearity in steel structures is due primarily to the effects of residual stresses and yielding. Different approaches by which these effects can be incorporated in the analysis will be briefly described in the following.

3.3.1. Allowance for residual stress effect on compression

The simplest approach to account for residual stress effect on compressive strength is to use the tangent modulus concept. The tangent modulus is defined as the slope of the stub column stress-strain curve shown in Fig. 2. If this curve is available, the tangent modulus E_t can be readily obtained as the slope of the stress-strain curve, i.e., $E_t = d\sigma/d\varepsilon$. If the stub column stress-strain curve is not available, an approximate value of E_t can be obtained as the ratio of the column strength in the inelastic range to the column strength in the elastic range. This approach is justified because most column strength curves were developed based on the tangent modulus concept. One expression for E_t, which is based on the Column Research Council (CRC) column strength curve, is

$$E_t = \begin{cases} E & \text{for } \dfrac{P}{P_y} < 0.5 \\ 4\left(\dfrac{P}{P_y}\right)\left(1 - \dfrac{P}{P_y}\right)E & \text{for } \dfrac{P}{P_y} \geq 0.5 \end{cases} \tag{2}$$

where P is the column axial force, P_y is the yield load and E is the modulus of elasticity.

Another expression for E_t, which is based on the Load and Resistance Factor Design (LRFD) column strength curve, is

$$E_t = \begin{cases} E & \text{for } \dfrac{P}{P_y} < 0.5 \\ -2.724\left(\dfrac{P}{P_y}\right)\ln\left(\dfrac{P}{P_y}\right)E & \text{for } \dfrac{P}{P_y} \geq 0.5 \end{cases} \tag{3}$$

The difference between the CRC column curve and the LRFD column curve is that the former does not consider member out-of-straightness in its derivation whereas the latter does. Consequently, the use of E_t in Eq. (3) implicitly includes the effect of member out-of-straightness in the analysis.

3.3.2. Allowance for residual stress effect on bending

The presence of residual stresses causes early yielding in members under flexure. Residual stresses also affect the cross-section M-Φ-P relationship. Mathematical expressions for M-Φ-P relationship of steel I-sections including the effect of residual stresses are available in the literature (see, for example, Chen and Atsuta 1976, and Liapunov 1974). These relationships can be used in a material nonlinear analysis if the effect of residual stresses on cross-sectional behavior is to be incorporated.

3.3.3. Allowance for yielding effect on bending

The effect of yielding on the flexural behavior of structural members can be accounted for in a number of approaches. These approaches can generally be classified into one the followings: (1) Elastic-plastic hinge method, (2) refined plastic hinge method, and (3) plastic zone method. The difference lies in the manner inelasticity is modeled, with elastic-plastic hinge approach being the most approximate and the plastic zone approach being the most accurate.

3.3.3.1. Elastic-plastic hinge analysis

This approach is also referred to as the concentrated plasticity approach. In this approach, inelasticity is assumed to concentrate in regions of plastic hinges. Other than at locations of plastic hinges, the member is assumed to behave elastically. Plastic hinges are locations where the internal moments M are equal to the cross-section plastic moment strength M_p, reduced for the presence of axial force if necessary. A number of expressions have been proposed to describe this cross-section moment-axial force interaction. The following give three such 2-D interaction equations for I-shaped hot-rolled sections.

• ASCE cross-section interaction equations (ASCE 1971):

- For strong axis bending

$$\frac{P}{P_y} + \frac{M}{1.18M_{px}} = 1 \qquad \text{for } 0.15 \leq \frac{P}{P_y} \leq 1.0 \tag{4a}$$

- For weak axis bending

$$\left(\frac{P}{P_y}\right)^2 + \frac{M}{1.19M_{py}} = 1 \quad \text{for } 0.40 \leq \frac{P}{P_y} \leq 1.0 \tag{4b}$$

where P is the axial force in the member, P_y is the yield load, M_{px} and M_{py} are the cross-section plastic moment strengths about the strong and the weak axes, respectively. If P/P_y falls below 0.15 for strong axis bending, or below 0.4 for weak axis bending, M is assumed to be equal to M_{px}, or M_{py}, respectively.

- Duan-Chen cross-section interaction equations (Duan and Chen 1989):
  - For strong axis bending

$$\left(\frac{P}{P_y}\right)^{1.3} + \left(\frac{M}{M_{px}}\right) = 1 \tag{5a}$$

  - For weak axis bending

$$\left(\frac{P}{P_y}\right)^{\beta} + \left(\frac{M}{M_{py}}\right) = 1 \tag{5b}$$

where $\beta = 2+1.2\, A_w/A_f$ in which A_w is the area of the web and A_f is the area of the flange. The other terms are as defined as before.

- Orbison, McGuire and Abel cross-section interaction equations (Orbison *et al.* 1982):
  - For strong axis bending

$$1.15\left(\frac{P}{P_y}\right)^2 + \left(\frac{M}{M_{px}}\right)^2 + 3.67\left(\frac{P}{P_y}\right)^2\left(\frac{M}{M_{px}}\right)^2 = 1.0 \tag{6a}$$

  - For weak axis bending

$$1.15\left(\frac{P}{P_y}\right)^2 + \left(\frac{M}{M_{py}}\right)^4 + 3\left(\frac{P}{P_y}\right)^6\left(\frac{M}{M_{py}}\right)^2 = 1.0 \tag{6b}$$

The terms in the above equations are as defined in Eqs. (4a) and (4b).

Frame analysis using the elastic-plastic hinge approach is normally carried out in a stepwise manner. When a plastic hinge is detected at a certain location, the stiffness matrix of the member containing the hinge will be modified. The process is repeated until the frame experiences instability or until a collapse mechanism forms when sufficient plastic hinges have formed. If geometrical nonlinearity is present, it is to be accounted for by the use of stability functions (in a beam-column formulation) or by the use of geometric stiffness matrix (in a finite element formulation). Computer software capable of performing this type of analysis is available to the general public (see for example, Sudhakar 1972, Chen and Sohal 1995). The approach has also been generalized for 3-*D* frame analysis (Powell and Chen 1986). The advantage of the elastic-plastic hinge approach is its ease of implementation. The disadvantage is its inability to model the effect of spread of plasticity. Although an elastic-plastic hinge analysis can generally capture the overall limit state behavior of structural frameworks and account for global force redistribution in a somewhat simplified manner, it can also give erroneous results especially for frames with columns that exhibit inelastic stability failure (Liew *et al.* 1993, 1994, White 1993). Because the results obtained are on the unconservative side, care must be exercised in basing the design on elastic-plastic hinge analysis results.

3.3.3.2. Refined plastic hinge analysis

The refined plastic hinge analysis, also known as the modified plastic hinge analysis, attempts to capture the effect of spread of plasticity in a simplified manner. The refined plastic hinge approach is aimed at modeling cross-section plastification, while ignoring spread of yield along member length. The gradual change in cross-section stiffness due to plastification as shown in the M-Φ-P curves in Fig. 3 can be modeled in a variety of ways. One approach is to modify the moment-rotational relationship of the member. If we denote ΔM_A, ΔM_B, $\Delta\theta_A$, $\Delta\theta_B$ as the incremental end moments and the end rotations at the ends A and B of a 2-D member respectively, the member moment-rotational relationship for a compact section member that does not experience any lateral torsional instability can be written as

$$\begin{Bmatrix} \Delta M_A \\ \Delta M_B \end{Bmatrix} = \frac{E_t I}{L} \begin{bmatrix} \left(s_{ii} - \frac{s_{ij}^2}{s_{ii}}\eta_B\right)(1-\eta_A) & s_{ij}(1-\eta_A)(1-\eta_B) \\ s_{ij}(1-\eta_A)(1-\eta_B) & \left(s_{ii} - \frac{s_{ij}^2}{s_{ii}}\eta_A\right)(1-\eta_B) \end{bmatrix} \begin{Bmatrix} \Delta\theta_A \\ \Delta\theta_B \end{Bmatrix} \tag{7}$$

where E_t is the tangent modulus, I is the moment of inertia, L is the member length, s_{ii}, s_{ij} are stability functions, and η_A, η_B are cross-section plastification factors at the ends A and B of the member, respectively. These factors are introduced to account for the effect of spread of yield in the cross-section. When η_i=0 (where subscript i denotes either A or B), the cross-section at i is fully elastic; when η_i=1, the cross-section is fully plastic; and when $0 < \eta_i < 1$, the cross-section is partially plastic. Various expressions have been proposed for η_i. Two such expressions are given in the following.

- Liew (1992)

$$\eta_i = \begin{cases} 0 & \alpha \le 0.5 \\ 1 - 4\alpha(1-\alpha) & \alpha > 0.5 \end{cases} \tag{8}$$

where α is a force-state parameter given by

$$\alpha = \begin{cases} \frac{P}{P_y} + \frac{8}{9}\left(\frac{M_i}{M_P}\right) & \text{for } \frac{P}{P_y} \ge \frac{2}{9}\frac{M_i}{M_P} \\ \frac{1}{2}\left(\frac{P}{P_y}\right) + \frac{M_i}{M_P} & \text{for } \frac{P}{P_y} < \frac{2}{9}\frac{M_i}{M_P} \end{cases} \tag{9}$$

in which M_i is the moment at cross-section i, and M_p is the cross-section plastic moment strength.

- King and Chen (1994)

$$\eta_i = \left(\frac{M_i - M_{yc}}{M_{pc} - M_{yc}}\right)^D \tag{10}$$

where M_i is the moment at cross-section i, $M_{yc}=(1-P/P_y)M_y$ is the yield moment adjusted for the presence of an axial force, M_{pc} is the plastic moment adjusted for the presence of an axial force, obtained by solving for M from Eq. (5), D is a decay factor given by

$$D = \begin{cases} 1.0 & \text{for } P/P_y < 0.2 \\ 0.8 & \text{for } 0.2 \le P/P_y < 0.6 \\ 0.6 & \text{for } 0.6 \le P/P_y < 1.0 \end{cases} \tag{11}$$

In a refined plastic hinge analysis, the incremental moment-rotation relationship of Eq. (7) is used in a matrix formulation for carrying out the frame analysis. Because of the nonlinear nature of the problem, iterative techniques are needed to perform the analysis. Detailed discussions of the refined plastic hinge approaches for 2-D frame analysis including verification studies and example problems can be found in Chen and White (1993), Chen and Toma (1994), and Chen and Kim (1997). PC-based computer programs using the aforementioned refined plastic hinge analysis technique have also been developed and are available to the general public (Chen and Toma 1994, Chen *et al*. 1996). More elaborate approaches based on the refined plastic hinge concept have also been formulated for 3-*D* frame analysis (Ziemian *et al*. 1992a, 1992b, Attala *et al*. 1993, Zhao 1993).

3.3.3.3. Plastic zone analysis

In plastic zone analysis, spread of inelasticity both within the cross-section and along the member length are modeled. Various levels of modeling to capture this spread of plasticity effect can be identified. At the highest level, a member is discretized into segments along its length and each cross-section is divided into small elemental areas (see for example, Vogel 1985, Ziemian 1990, Taucer *et al*. 1991, Clarke 1994, among others). The effects of geometrical nonlinearity, residual stresses, geometric imperfections, lateral torsional instability, local instability, etc., are explicitly modeled. Resultant forces and moments are obtained by numerical integration of elemental stresses over the cross-section, and member force-deformation relationship is obtained by numerical integration of force and kinematic quantities along the member length. Needless to say, such analysis is extremely cumbersome and time consuming to perform, making it quite undesirable and formidable for routine use. The analysis is usually performed for purpose of research, validation studies of special or unusual structures, and for providing benchmark numerical data against which other simplified procedures can be checked.

Over the years, a number of simplified approaches to plastic zone methods of frame analysis have been proposed. The simplifications range from eliminating the need to perform cross-section stress integration by employing an established cross-section M-Φ-P relationship (Lui 1990, 1992) to defining an effective member stiffness based on stiffness properties of selected points along the member length (Li and Lui 1995). Other approaches include the use of a force-based interpolation function to describe the variation of cross-section internal forces as part of the basis for the inelastic element formulation (Attalla 1995), and consistent linearization (Marsden and Hughes 1983) of the nonlinear finite element equations to obtain the element stiffnesses (Nukala 1997). Like the refined plastic hinge approaches, these simplified plastic zone approaches require the use of special programs to carry out the analysis. However, because of the explicit modeling of geometrical and material nonlinearities, the results obtained are quite accurate.

4. Notional load approach

Notional loads are fictitious lateral loads apply to a frame to simulate geometrical imperfection and/or inelastic effects. The basic concept of notional load is depicted in Fig. 8(a) in which a cantilever beam-column with an initial out-of-plumbness of Δ_o is subjected to a horizontal force H and a vertical force P. If Δ is the additional lateral deflection experienced by the member, its base moment evaluated based on the deformed geometry is $HL+P(\Delta_o+\Delta)$. This same moment can be obtained if a notional lateral load of magnitude $P(\Delta_o+\Delta)/L$ is applied in conjunction with the actual lateral load to a geometrically perfect member in its undeformed state as shown in Fig. 8(b). In general, this notional load can be expressed as ξP, where ξ is a coefficient, which when multiplied by the gravity load P, gives the correct second-order moment in the member.

Although the concept of notional load is relatively simple to comprehend, the determination of the value of ξ is not a simple matter. This is because its value depends on a number of factors. Among these are the magnitude and distribution of applied lateral and gravity loads as well as frame geometry and

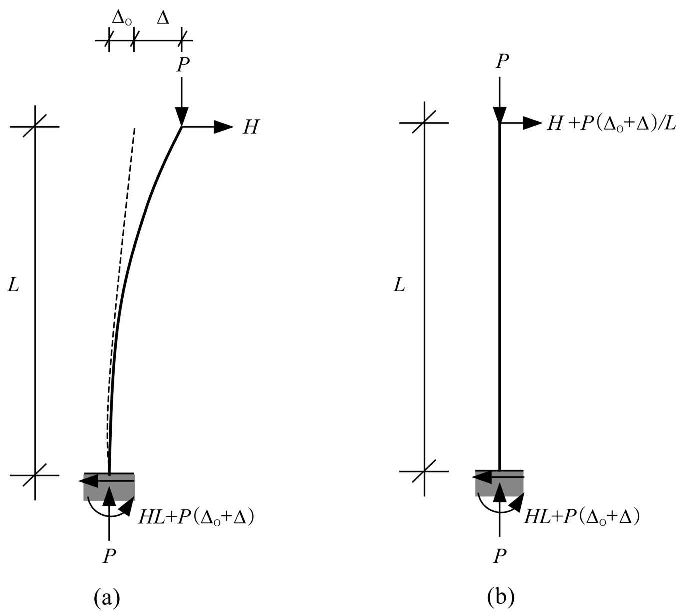


Fig. 8 Notional load concept

member stiffness. Note that member stiffness (and hence frame stiffness) changes when inelasticity sets in. For simplicity, ξ is often obtained through calibration with existing design equations (Liew *et al.* 1994, ASCE 1997).

5. Member/frame interaction

The current design practice for steel frames is based on satisfying individual member rather than overall system capacity. Nonetheless, the load-carrying capacity of a member is closely tied to its interaction with other members of the frame. This interaction effect can be accounted for by the use of either the effective length or the notional load approach (ASCE 1997).

In the effective length approach, an effective length KL (where K is the effective length factor and L is the true length) is used to compute the nominal axial strength of the beam-column under consideration. The effective length of a member is the length of a fictitious pinned-pinned compression member whose load carrying capacity is equivalent to that of the actual member considering member/frame interaction effect. Because $K \geq 1$ for a typical compression member in a frame subject to sideways, the nominal axial member strength computed using its effective length will be smaller than or at most equal to its strength computed using the actual member length. Over the years, various equations for K have been proposed to take into consideration the various forms of member/frame interaction effects on member strength. Some of these include joint flexibility, inelasticity, and leaning column effects.

In the notional load approach the actual member length (i.e., K=1) is used to calculate the beam-column's nominal axial strength. As a result, the axial force term in the interaction equation (to be discussed in the next section) will be smaller. In order to obtain a comparable value for the interaction equation, notional loads are applied to the frame to increase the value of the flexural term of the interaction equation.

In other words, any 'gain' in the axial force term is compensated by a 'loss' in the flexure term so that no change in overall value of the interaction equation will result.

6. AISC LRFD approach for steel frame design

In this section a succinct summary of the approach recommended by the AISC LRFD specification (LRFD 1999) for incorporating geometrical and material nonlinearities as well as geometric and material imperfections in steel frame design is given. Because the current design philosophy is based on the satisfaction of individual member rather than overall system capacity, the design is considered satisfactory if all members of the frame satisfy certain member capacity requirements. To carry out the member capacity check, the frame is first analyzed to obtain the required strength for each member. This required strength is then checked against the member design strength. If the required strength does not exceed the design strength, the strength design criterion is said to be satisfied. Design strength equations for various types of members (e.g., tension members, compression members, flexural members, etc.) are given in the specification. Most of them are developed by semi-empirical means, which involve the use of some basic mechanics theories in conjunction with available numerical and experimental data. Very often, some forms of statistical and curve-fitting techniques are also employed.

6.1. Geometrical nonlinearity

Geometrical nonlinearity is accounted for either (1) directly by performing a second-order elastic analysis, or (2) indirectly by using moment magnification factors (B_1 and B_2) in lieu of a second-order elastic analysis. In using the moment magnification approach, two first-order analyses are performed on the frame. In the first analysis, the frame is artificially prevented from sway (by providing fictitious supports at each story level) and analyzed for gravity loads. In the second analysis, the frame is allowed to sway (by removing the fictitious supports) and analyzed for any applied lateral loads in conjunction with the reactions (applied in a reverse sense to the frame) from the fictitious supports. The maximum moments obtained for each member from these two analyses, denoted as M_{nt} and M_{lt}, respectively, are then combined to obtain the required flexural strength M_u for the member using the equation

$$M_u = B_1 M_{nt} + B_2 M_{lt} \tag{12}$$

where B_1 is the P-δ moment magnification factor, and B_2 is the P-Δ moment magnification factor given by (LRFD 1999)[†]

$$B_1 = \frac{C_m}{1 - \dfrac{P_u}{P_{e1}}} \geq 1, \quad B_2 = \frac{1}{1 - \sum P_u \left(\dfrac{\Delta_H}{\sum HL} \right)}, \text{ or } B_2 = \frac{1}{1 - \dfrac{\sum P_u}{\sum P_{e2}}} \tag{13a,b,c}$$

in which C_m is a factor to account for moment gradient in the member, P_{e1} is the Euler buckling load ($=\pi^2 EI/L^2$), P_u is the required axial compressive strength of the member, ΣP_u is the required axial compressive strength of all columns in a story, Δ_H is the lateral inter-story deflection due to story shear ΣH, L is the story height, P_{e2} is the sway buckling load of the member ($=\pi^2 EI/(KL)^2$), K

[†]In the draft LRFD Specification (LRFD 2003), a factor of 0.85 is applied to the term ΣHL for B_2

is the effective length factor evaluated in the plane of bending, ΣP_{e2} is the sum of P_{e2} of all columns in the story, and EI is the flexural rigidity of the member in the plane of bending.

The use of Eq. (12) presupposes that the maximum effects due to member instability and frame instability coincide at a point when in reality the maximum P-δ moment usually occurs between member ends and the maximum P-Δ moment often occurs at the member ends. However, the moment magnification approach will give reasonable accurate results for ordinary moment frames that exhibit more or less a shear mode of deformation and for frames that do not exhibit significant interstory interaction.

6.2. Material nonlinearity

Material nonlinearity is often accounted for implicitly in the design equations. For instance, the column strength, the beam strength and the beam-column strength design equations were all derived using a semi-empirical approach in which extensive use was made of numerical and/or experimental data generated for the specific type of structural members loaded into the inelastic range. The only exception to this is the explicit use of the tangent modulus E_t in calculating G factors when inelastic effective length factor $K_{inelastic}$ is used in column and beam-column design.

The implicit consideration of material nonlinearity eliminates the need for a material nonlinear analysis. However, it should be noted that forces and moments obtained from an elastic analysis are likely to be different from those obtained from an inelastic analysis. This inconsistency of using elastic results for the design of inelastic structures has not been adequately addressed in the current design provisions.

6.3. Geometric and material imperfections

Like material nonlinearity, geometric and material imperfections are implicitly considered in the design equations. For instance, the LRFD column equation was derived from curve-fitting of the compressive strengths of initially crooked pinned-pinned columns. The initial crookedness was assumed to be $L/1500$ where L is the column length (Bjorhovde 1972, Tide 1985). In cases when the effective length factor is used to account for the effect of member-frame interaction, out-of-plumbness is also tacitly accounted for. For example, the design of a cantilever column using the column equation with a *theoretical* K factor of 2 tacitly assumes that the column has an initial out-of-plumbness of $L/1500$. To account for the residual stress effect on early yielding, the column load that demarcates inelastic from elastic behavior is set at $P/P_y = 0.39$. When $P/P_y > 0.39$, the column is assumed to experience inelastic buckling.

Although geometric imperfection is not accounted for in the LRFD beam design equation for beams experiencing elastic lateral torsional instability, such imperfection is implicitly accounted for in the design equation for beams experiencing inelastic lateral torsional instability. This is explained by the fact that while the design equation for elastic lateral torsional instability is based on the theoretical buckling moment of a geometrically perfect beam, the design equation for inelastic torsional instability is an empirical equation based almost entirely from curve-fitting of experimental data. Since geometric imperfections are inherent in the test beams, geometric imperfection is therefore implicitly accounted for in this design equation. To account for the residual effect on early yielding, the theoretical yield moment M_y is reduced by the term $F_r S_x$ when the beam is bent about its major axis, where F_r is the compressive residual stress in the beam flange, taken as 10 ksi (69 MPa) for rolled shapes and 16.5 ksi (114 MPa) for welded shapes, and S_x is the section modulus.

6.4. Beam-column interaction equations

The adequacy of a member to resist the combined effect of flexure and axial force is checked against one of the following two interaction equations

$$\frac{P_u}{\phi_c P_n}+\frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}}+\frac{M_{uy}}{\phi_b M_{ny}}\right)\le 1.0, \qquad \text{for } \frac{P_u}{\phi_c P_n}\ge 0.2 \tag{14a}$$

$$\frac{P_u}{2\phi_c P_n}+\left(\frac{M_{ux}}{\phi_b M_{nx}}+\frac{M_{uy}}{\phi_b M_{ny}}\right)\le 1.0, \qquad \text{for } \frac{P_u}{\phi_c P_n}< 0.2 \tag{14b}$$

where P_u, M_{ux} and M_{uy} are the required axial, and flexural strength about the member's x and y axes, respectively. P_n, M_{nx} and M_{ny} are the corresponding nominal axial, and flexural strengths (LRFD 1999). ϕ_c (=0.85[††]) and ϕ_b (=0.90) are the resistance factors for compression and bending, respectively.

The beam-column interaction equations were derived from strength interaction curves reported by Kanchanalai (1977). Although the initial Kanchanalai curves include only the effect of residual stresses but not the effect of geometric imperfection, these curves were subsequently adjusted for the presence of geometric imperfections by LeMessurier (Liew *et al.* 1991), curve-fitted and linearized to produce the two interaction equations. The beam-column design equations thus implicitly account for both geometric and material imperfections.

6.5. Direct analysis method

A direct analysis method that makes use of the concepts of notional load and reduced flexural stiffness discussed earlier in this paper to account for geometrical imperfections and inelasticity is proposed for the design of moment frames in the draft LRFD Specification (LRFD 2003). In this approach, the frame is analyzed by:

(1) applying a notional load in addition to any factored lateral load at story i. The magnitude of this notional load is taken as 0.2% of the gravity load acting at that story, and

(2) using a reduced flexural rigidity EI^* for all members in the frame, where

$$EI^* = 0.8E_t I \tag{15}$$

in which I is the moment of inertia of the member and E_t is the tangent modulus calculated using Eq. (2). Alternatively, one can use E_t=E (i.e., the elastic modulus) for all members with the provision that when $P_u > 0.5P_y$, the magnitude of notional load used is increased from 0.2% to 0.3% of the gravity load.

The required axial force and flexural strengths obtained from a second-order elastic analysis, or through the use of B_1 and B_2 moment magnification factors in conjunction with a first-order elastic analysis, are substituted in the appropriate interaction equation (but with P_n calculated using an effective length factor of unity) to check for the adequacy of the member to resist the combined effects of axial force and bending moments.

[††] ϕ_c=0.90 is proposed in the draft LRFD Specification (LRFD 2003)

7. Examples

In this section three example frames will be used to compare the current and proposed AISC LRFD approaches in designing moment frames. In the first example, the capacity of the leeward beam-column of a simple portal frame with geometric properties shown in Fig. 9(a) calculated using Eqs. (14a) or (14b) in conjunction with the current and the proposed (i.e., direct analysis) procedures but without the use of the resistance (ϕ) factors is shown in Fig. 9(b) in the form of an interaction diagram. The end restraint factor at joint i is defined as $G_i=(I_c/L_c)/(I_b/L_b)_i$. As can be seen, both approaches compare well with the Kanchanalai (1997) results adjusted for the presence of geometrical imperfections, with the direct analysis method giving slightly conservative results.

In the second example the capacity of the leeward column of a frame with a leaning column as shown in Fig. 10(a) calculated using the current and the proposed AISC LRFD procedures is compared in Fig. 10(b) to the Kanchanalai results. As can be seen, both approaches give conservative results and are

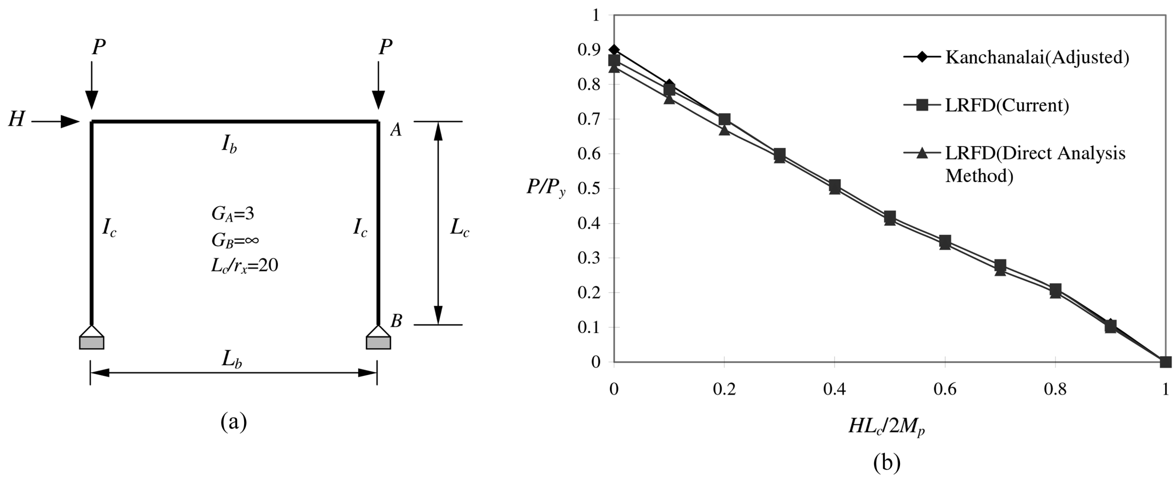


Fig. 9 Comparison of load-carrying capacity of column AB for a simple portal frame

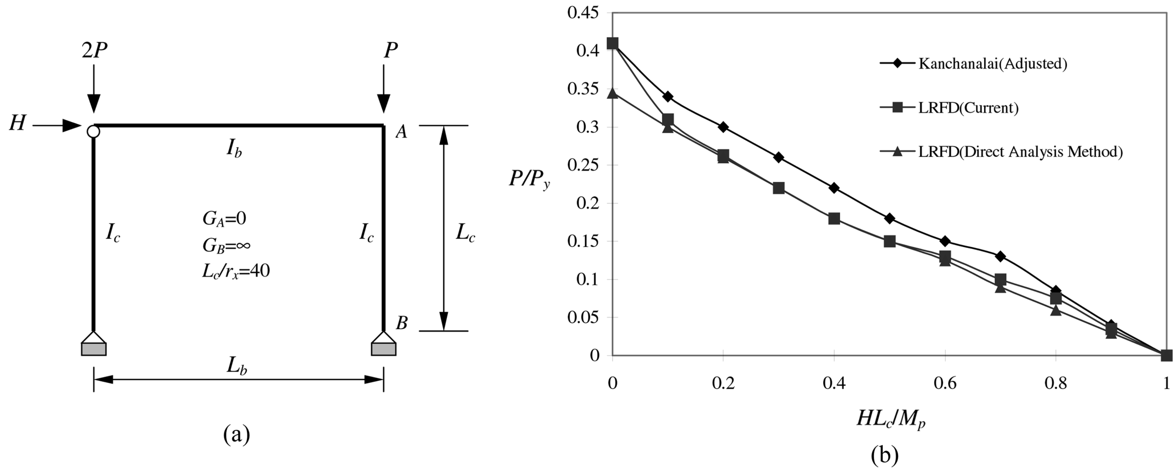
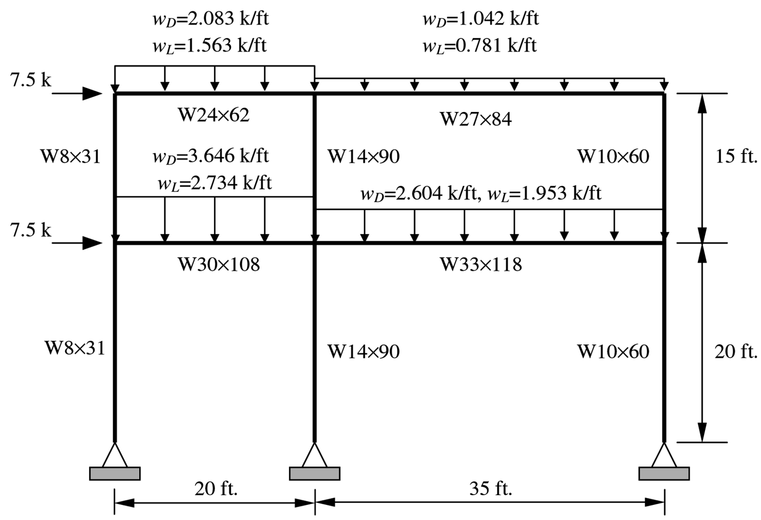


Fig. 10 Comparison of load-carrying capacity of column AB for a simple portal frame with a leaning column



All loads shown are service (unfactored) loads
(1 ft. = 0.305 m , 1 k/ft. = 14.6 kN/m)

Fig. 11 A two-story two-bay moment frame

Table 1 Comparison of results for the interaction equation check

Member	Gravity (1.2*D*+1.6*L*)		Gravity+Wind (1.2*D*+0.5*L*+1.6*W*)	
	LRFD (Current)	LRFD (Direct Analysis)	LRFD (Current)	LRFD (Direct Analysis)
Leeward	0.494	0.523	0.886	0.935
Middle	0.510	0.456	0.901	0.957
Windward	0.736	0.697	0.821	0.858

comparable except in the region where the axial load is high and the bending moment is low. The conservatism in this region is due to the presence of the notional load, which creates moment in the leeward column that is magnified by the high axial load acting on the frame.

In the third example the comparison between the current and the proposed AISC LRFD procedures was made for a two-story two-bay frame shown in Fig. 11 for two load combinations (gravity, and gravity+wind). The frame was a slight modification from the one given in ASCE (1997). The values computed using the terms on the left hand side of the controlling interaction equation for the three first-story beam-columns are summarized in Table 1. For this particular frame, design is controlled by the gravity+wind load condition. For this load case, it can be seen that the direct analysis procedure gives a more conservative design when compared to the current procedure. However, this conservatism is by no means excessive.

8. Conclusions

In this paper, various behavioral aspects that affect the strength and stability design of steel moment frames are presented, and manners in which they are accounted for in analysis and design are also discussed. A new LRFD design procedure refers to as the direct analysis method that makes use of the notional load concept and reduced flexural rigidity was outlined.

Regardless of the method used, it should be noted that the current design philosophy for steel frames is based on the use of an interaction equation applied to each beam-column of the frame on a member-to-member basis. As a result, except for the design of low-rise frames using the plastic design provisions (ASCE 1971) or the design of continuous beams, force redistribution within the structural system is ignored. The use of this so-called first hinge limit state concept in the design of moment frames essentially neglects any force redistribution effect that is inherent in a highly redundant structure. The resulting design is often conservative, but uneconomical, because no consideration is given to account for the ability of the structure to seek alternate load paths when a member fails. Another drawback of the current approach is that elastic analysis is used to gauge the response of the structure in the inelastic range. This discrepancy between what is assumed in the analysis and what the structure is actually experiencing represents a major inconsistency in a limit states design because failure in a ductile structure such as a steel frame is often preceded by or associated with significant yielding.

Direct allowance for force redistribution in the design of multistory steel frames often necessitates the use of advanced analysis techniques (such as the modified plastic hinge and plastic zone approaches discussed earlier) and special frame analysis software. Even though such techniques are not widely understood nor commonly used, they are readily available in the literature. In addition, user-friendly software capable of performing second-order inelastic analysis is becoming more and more accessible (see for example McGuire *et al.* 2000). With a better understanding of the inelastic behavior of structures through research and education, coupled with the availability of user-friendly advanced analysis software and awesome computational power of personal computers, it is anticipated that a system approach to steel frame design will be realized in the foreseeable future. In this approach, all important parameters that significantly affect frame response from start of loading to failure are to be considered in the analysis. The frame is to be designed as an entity rather than as a collection of members. A satisfactory design is one in which the frame performs as intended without experiencing any premature failures. This holistic approach to design not only allows the full capacity of the frame to be realized, but is more in line with the performance based design concept currently in use in the field of earthquake resistant design.

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