Proceedings of the
Annual Stability Conference
Structural Stability Research Council
St. Louis, Missouri, April 16-20, 2013

System reliability of steel frames designed by inelastic analysis

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Abstract
The paper is concerned with the design of steel frames under gravity loads by geometric and material nonlinear analysis, also referred to as "inelastic analysis" in Appendix 1 of the ANSI/AISC360-10 Specification. In this approach, the strength of a structural frame is determined by system analysis in lieu of checking member resistances to the specific provisions of the Specification for tension, compression, flexural members etc., provided a comparable or higher level of structural reliability by the analysis. In this paper the reliability of steel frames is evaluated by performing Monte Carlo simulations for a series of 2D low-to-mid-rise moment resisting frames, including regular and irregular configurations. The analyses treat the material properties, initial geometric imperfections, residual stresses and loads as random variables and suggest suitable system resistance factors for different system reliability levels. Member cross-sections are selected in a way to provide different system failure modes such as sway instability and/or member failure. In designing by inelastic analysis, the system resistance factor (φ_s) is applied to the frame strength determined by analysis, and provided the reduced system strength exceeds the loads, the design is deemed adequate, requiring no further check of individual member resistance. The procedure is more efficient than current procedures based on elastic analysis and provides the designer with a greater understanding of the behavior of the frame. It promotes a more holistic approach and greater innovation in structural design and is likely to become increasingly used by structural engineers as commercial software packages increasingly make geometric and material nonlinear analyses available.

1. Introduction

In conventional steel design procedure the members such as beams, columns and connections are isolated from the structural system and designed individually based on an LRFD (Load Resistance Factor Design) format:

$$\varphi R_{ni} \geq \sum \gamma_i Q_{ni} \tag{1}$$

in which R_{ni} is the *i*th member capacity calculated based on steel design code and Q_{ni} is the load applied to the corresponding member. In this approach, the interaction between the structural

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system and its members is only reflected through the use of effective length factor (k). However, this component-based approach cannot accurately capture the influence of the inelastic redistribution of internal forces. On the other hand, the interaction between the members, specially in a large structural system, is too complex to be represented by the simple effective length factor (Chen and Kim 1997). Thus, this design methodology may not accurately predict the ultimate load-carrying capacity of structural systems or the frame failure modes. There are strong economic and safety reasons to develop a practical method that can account for compatibility between the members and the whole system.

As an alternative, "advanced" second order inelastic analysis (or in European terminology "GMNIA, Geometric and Material Non-linear with Imperfections Analysis") represents a new method in which analysis and design is integrated together in a single step. The proposed system strength check has the LRFD type format:

$$\varphi_s R_n \geq \sum \gamma_i Q_{ni} \tag{2}$$

in which R_n is the nominal system strength predicted by inelastic analysis and φ_s is the system resistance factor determined by reliability assessment. It is worth to mention that although Eq. 2 has the same format as LRFD, it follows a different philosophy as it is based on system performance. Since member failures are directly incorporated into advanced analysis, there is no need for separate member/section capacity check based on a design specification.

Advanced analysis is now permitted by several steel structure design codes (AS4100 1998; AISC360-10 2100; Eurocode 3 2005). With the rapid development of computer software there is no longer a barrier to use this type of analysis in practical applications. A great attention has been devoted to the research on advanced analysis of steel structures over the past 20 years (Ziemian 1990; Liew, White et al. 1993; Chen and Kim 1997). Fig. 1 schematically shows that among all analysis methods, advanced analysis can accurately predict the behavior and ultimate load carrying capacity of a structural system, taking into account system effects explicitly such as load redistribution subsequent to first yielding.

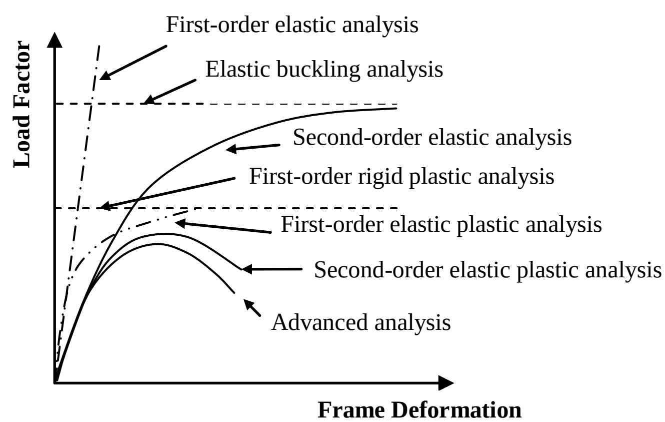

Figure 1: Structural analysis methods

By using advanced analysis the system failure mode becomes apparent and it is possible to consider the consequences of failure in the design process. Ziemian, et al. (1992) analysed a series of two-bay, two-story planar frames and a 22-story, 3D frame and showed that design by advanced analysis could save about 12% of steel weight compared to design by LRFD specification. The main problem is that the Specifications require the reliability of the system to be considered but do not explain how this may be achieved. On the other hand if advanced analysis is used to determine the ultimate strength of the whole system, the main difficulty is to assign an appropriate system resistance factor (φ_s) which accounts for the main factors influencing the reliability of the frame. This paper is part of the research effort to determine the appropriate system resistance factors for different types of structural systems. In the present paper only gravity loads were considered.

2. Methodology

The procedures of developing a system reliability-based design format for steel frames can be summarized in five essential steps as follows:

(1) A series of low-to-mid-rise steel frames are chosen. Every frame is first designed based on (AS4100 1998) as a starting point.

(2) Different system resistance factors (φ_s) between 0.6 and 1 are assumed for each frame and the frame is modified to satisfy the limit state equation (Eq. 2) (system-based design). This can be achieved by adjust either the cross-sections or the loads. In the former, for each specific value of φ_s, new combinations of cross-sections are selected to satisfy the limit state equation while the total applied load remains constant (referred to as Method 1 in this paper). The second approach is based on changing loads for different value of φ_s to satisfy Eq. 2 while the cross-sections remain unchanged (Method 2 in the following discussion).

(3) For all designed frames, Monte Carlo simulations are performed to develop a probabilistic model (distribution type, mean and standard deviation) for the system strength, considering the randomness in material and geometric properties.

(4) Using the developed statistics for the frame ultimate strength (R) in Step 3, and the probabilistic models for dead load (G) and live load (Q), the reliability index (β) can be determined for all frames by first order reliability analysis (FORM) (Melchers 1999). The reliability index relates the structural failure probability by $P_f = \Phi(-\beta)$, where P_f is the probability of failure ($P(R - G - Q \leq 0)$ and Φ() is the standard normal distribution function. Different live to dead load ratios (Q_n/G_n) are considered.

(5) For different frames with different failure modes, the relationships of β (reliability index) versus φ_s (system resistance factor) are plotted and φ_s can be obtained for different levels of target reliability.

3. Analytical model

A series of 2D steel frames, which presents common Australian steel building structures, have been selected as a basis for the present study. Fig. 2 shows the geometry, support conditions and loading pattern. The design of these frames is controlled by gravity loads, therefore the load combination of $1.2G + 1.5Q$, which is based on Australian standard (AS4100 1998), is applied to the frames. The total gravity load is applied as Uniform Distribution Load (UDL) along the beam lengths. The nominal dead and live loads are assumed to have a same magnitude of w=25 kN/m. These frames are first design based on AS4100 (1998) and member cross-sections are presented in Table 4.

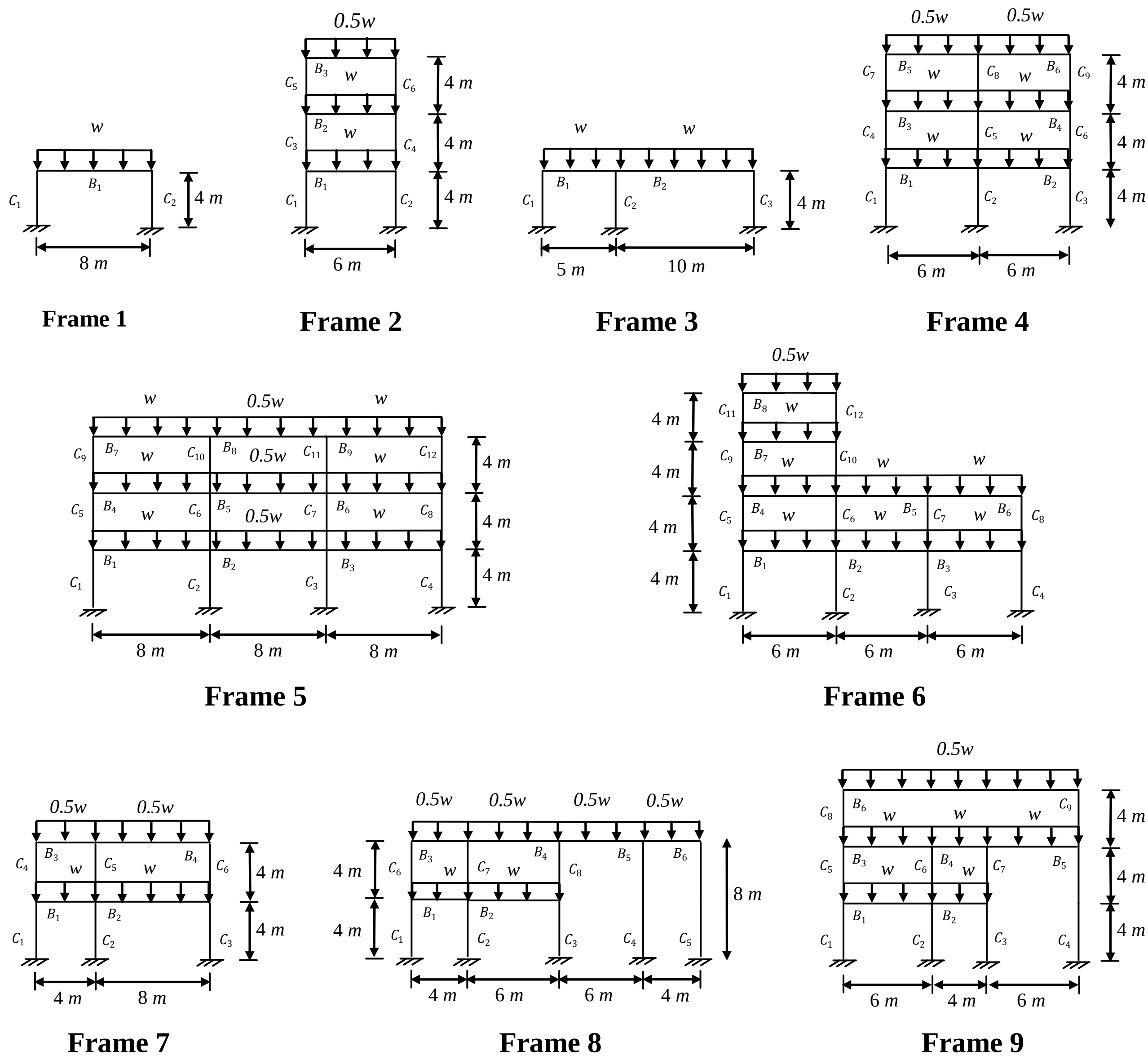

Figure 2: Layouts for steel framing system

Two dimensional second-order inelastic FE models are developed for each frame using the commercial finite element (FE) software ABAQUS (2009), accounting for all material and geometrical nonlinearities. To model the material nonlinearity, the 2D plastic-zone beam-column element is used to trace the spread of plasticity through the cross-section and along the member length. Using incremental load deflection response, the element geometry in each load increment is updated and the second-order effects can be captured. The material is modeled as elastic-perfectly-plastic with elastic modulus (E) equal to 200 GPa and yield strength of 320 MPa. All column bases are fully fixed and the joints are modeled as rigid. Residual stress is modeled as self-equilibrium initial stress using ECCS model (ECCS 1984) and defined at default cross-section integration points of ABAQUS. A FORTRAN subroutine is written to implement residual stress into finite element models. Initial geometric imperfection is modeled as linear superposition of the first six buckling modes. More details about this method and appropriate scale factor of each mode can be found in Shayan, et al. (2012). A mesh convergence study is performed and one element per 200 mm length is used for all members. All the cross-sections are fully compact and the out-of-plane behavior is restrained (2D frames). Thus, local buckling and lateral-torsional buckling are not considered in this study.

As mentioned by Galambos (1990) the structural system may not fail when one element fails. Perhaps different combinations of element failures may take place to reach the point in which the failure of whole structural system happens. So, it is important to consider different system failure modes. Beam/beams as well as column/columns can be fully or partially yielded and the combination of these failures is considered. If any beams or columns cross-sections are yielded more than 75%, they are categorized as fully yielded and they are referred as BFY (beam fully yielded) and CFY (column fully yielded) in this study. If the yield ratio in any beams or columns is less than 75%, they are categorized as beam partially yielded (BPY) and column partially yielded (CPY), respectively. Frame ultimate load is defined as the maximum point of the load-displacement curve or the point when the storey drift exceeds 5%, whichever comes first.

4. Probabilistic analysis

Latin Hypercube Sampling (LHS) is conducted to drive the statistical information of the frame strengths. Compared to the direct random sampling, LHS requires less samples to achieve similar accuracy. In the present study, 350 advanced analyses were performed for each frame using randomly generated values for yield stress, elastic modulus, cross-sectional properties, member and frame initial geometric imperfections and residual stress, as described in Section 4.1. To determine the statistics of frame strength, the frames are loaded with an increasing nominal unfactored gravity load ($G + Q$) until structural collapse. The dead and live loads are not treated as random variables in this stage and randomness in loads will be considered later in finding the probability of failures and thereby the reliability index.

4.1 Uncertainties in steel structures

In construction practice, the steel members are characterized by inherent high variations of geometric and material properties. Various sources of uncertainties exist in steel structures and can influence the load-carrying capacity of a structure. The basic random variables considered in this study are: yield stress (F_y), residual stress, elastic modulus (E), cross-sectional properties such as flange width (b_f), web height (h), and flange and web thickness (t_f and t_w), member out-of-straightness (δ) and frame out-of-plumb (Δ).

4.1.1 Variability in yield stress and elastic modulus

The yield stress is one of the most important characteristics of steel structures which often has a great influence in load-carrying capacity of the whole system. In this study the yield stress is modeled as a lognormal distribution with the mean of $1.05\mathrm{F}_{\mathrm{yn}}$ and a coefficient of variation (COV) of 0.1 provided by Galambos and Ravindar (1978). Here, F_{yn} is the nominal yield strength of steel considered as 320 Mpa. It is assumed that the yield strength is perfectly correlated between all members and one random F_y is generated and assigned to all beams and columns. The perfectly correlated case is of interest because of its similarity to deterministic analysis where all members are assigned a single nominal yield strength (Buonopane and Schafer 2006).

The modulus of elasticity is modeled as a normal distributed variable with a mean equal to the nominal value (200 GPa) and a COV of 6% (Galambos and Ravindar 1978). Again perfectly correlated case has been evaluated.

4.1.2 Variability in cross-section geometries

Geometrical section properties are statistically evaluated by Melcher, et al. (2004), based on the experimental measurement of 369 hot-rolled I-sections. The relative geometrical characteristics are listed in Table 1 as the ratio of the real characteristic obtained from the measurement of cross-sectional geometry (e.g. cross-section depth h) to the nominal dimension of the cross-sectional one. Strong correlations have been observed between all measured data and also employed in this study. The correlation matrix can be found in Eq. 3.

Table 1: Statistical result of geometrical characteristics

Thickness	Mean	Standard deviation
Section depth (h)	1.001 h	0.00443
Section width (b_1)	1.012 b_1	0.01026
Section width (b_2)	1.015 b_1	0.00961
Web thickness (t_1)	1.055 t_1	0.04182
Flange thickness (t_{21})	0.988 t_{21}	0.04357
Flange thickness (t_{22})	0.988 t_{22}	0.04803

$$\begin{bmatrix} 1 & -0.0068 & 0.0534 & 0.0399 & -0.0686 & -0.0989 \\ -0.0068 & 1 & 0.6227 & -0.2142 & -0.2681 & -0.1456 \\ 0.0534 & 0.6227 & 1 & -0.2132 & -0.1596 & -0.0423 \\ 0.0399 & -0.2142 & -0.2132 & 1 & 0.2368 & 0.2451 \\ 0.0686 & -0.2681 & -0.1596 & 0.2368 & 1 & 0.7634 \\ -0.0989 & -0.1456 & 0.0423 & 0.2451 & 0.7634 & 1 \end{bmatrix} \tag{3}$$

Using the statistics in Table 1 and correlation matrix in Eq. 3, the mean and COV of cross-sectional area can be obtained as 1.025A and 0.032 respectively which is comparable with the statistical data reported in other papers (Strating and Vos 1973; Fukumoto and Itoh 1983).

4.1.3 Variability in initial geometric imperfection

Modeling imperfection is based on the method proposed by Shayan, et al. (2012). To model initial out-of-straightness as random quantity, experimental measurements of nine IPE 160 columns carried out at the University of Politecnico di Milanoand and published by ECCS Committee 8.1 (Sfintesco 1970), are used. The reported data comprises geometric imperfection measurements at mid-length and quarter points. All the members are hot-rolled, simply-supported and axially loaded. The actual non-dimensionalised measured imperfection is then extracted into first three buckling modes (Fig. 3 a) and the statistical characteristics mean (μ), standard deviation (σ) and probabilistic distribution of different modes are acquired (Table 2).

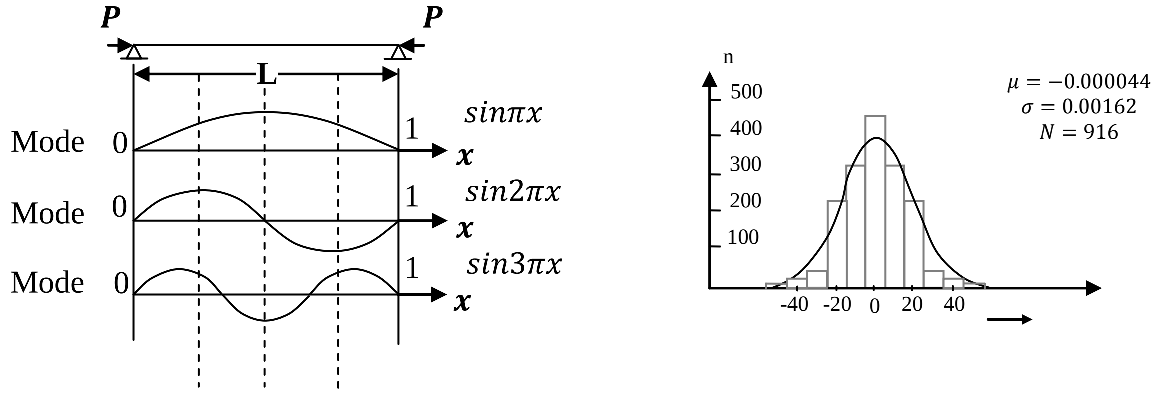

Figure 3: (a) Buckling modes of simply supported, axially loaded column, (b) Out-of-plumb statistics

Table 2: Statistic characteristics of scale factors

Statistics	a_1	a_2	a_3
Mean (μ)	0.000556	0.000139	0.000073
Standard deviation (σ)	0.000427	0.000071	0.000078
Distribution	Normal	Normal	Normal

Using these characteristics, the random shape of imperfection can be determined for each single member of a steel frame by generating *k* random scale factors for each mode and combine them with assigning a random sing to each mode. Additionally random sway imperfection (out-of-plumb), is generated for whole frame using the statistics provided by Lindner (1984) (Fig. 3 b) and is superimposed into FE model.

4.1.4 Variability in residual stress

To consider the variability of residual stresses total of 63 actual measurements are obtained from the literature (Shayan, et al. 2013). First the measured residual stresses at *i* nodes distributed in each cross-section are non-dimensionalised by dividing the stresses by the value of reported yield stress (F_y) denoted by σ_{exp} (Fig. 4 b). Then, the corresponding theoretical non-dimensional residual stresses are calculated at the same points using ECCS (1984) distribution (Fig. 4 a).

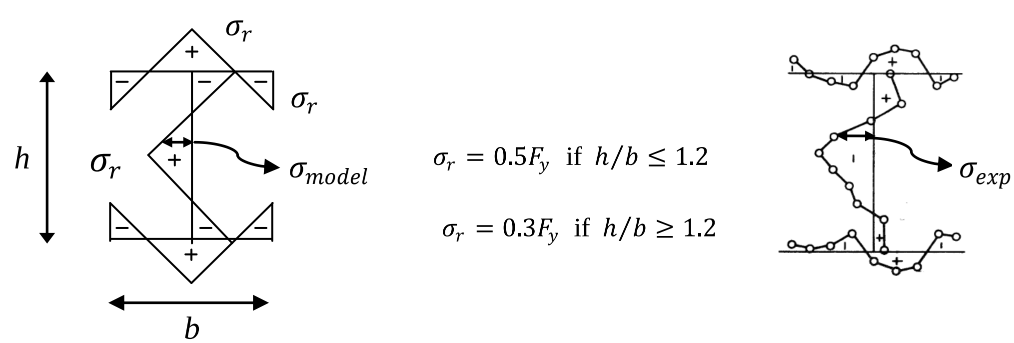

Figure 4: Residual stress (a) ECCS model, (b) Example of measured residual stress

Finally, the scale factors which minimize the error between theoretical models and experimental measurements are derived by error minimization. The error is defined as:

$$Error = \sum_{i=1}^{n}(X\sigma_{model}^{i} - \sigma_{exp}^{i})^2 \quad (4)$$

in which n is total number of measurements for each cross-section. The error minimization is then performed (Eq. 5) and the scale factors (X) for all are obtained:

$$\partial Error/\partial X = 0 \quad (5)$$

The scale factors are normally distributed with a mean of 1.047X and COV of 0.21 where X is the scale factor applied to ECCS residual stress model. The residual stress is assumed to be constant along the length of the member and correlated between the members of a frame. More details can be found in Shayan, et al. (2013).

5. System reliability

The basic structural reliability is to find the probability of failure (P_f) of the structure, defined as:

$$P_f = P(g(X) \leq 0) = \int \dots \int_{g(X)\leq 0} f_{\mathrm{X}}(\mathrm{x})\, d\mathrm{x} \quad (6)$$

in which P_f is the probability of failure of the structure, $X = (X_1, \dots . X_n)$ is the n-dimensional vector of the random variables such as applied load and structural system resistance, $f_{\mathrm{X}}(\mathrm{x})$ is the joint probability density function for X, $g(X)$ is the limit state function and $g(X) \leq 0$ defines the unsafe (failure) region. The classical structural reliability equation (Eq. 6) is transformed to the more practical and familiar format of LRFD to use in advanced analysis and design equation is defined as:

$$\varphi_s R_n = 1.2G_n + 1.5Q_n \quad (7)$$

In this equation, G_n and Q_n are the total nominal dead and live load respectively. The system reliability index (β) can be estimated using the first order reliability method (FORM) (Melchers 1999), with the simple limit state function $g = R - G - Q$, in which R is system resistance or frame ultimate strength, G is dead load and Q is live load. The limit state can be rearranged as:

$$g = \frac{R}{R_n} \times \left(\frac{1.2+1.5(Q_n/G_n)}{\varphi_s}\right) - \frac{G}{G_n} - \frac{Q}{Q_n} \times \frac{Q_n}{G_n} \quad (8)$$

When the actual distributions of the random variables are taken into account, including distributions which are non-normal, the first order reliability method (FORM) is performed in an iterative manner. The mean-to-nominal strength ($\bar{R}/R_n$) (bias) statistics are determined from the frame simulations while the statistics of loads can be obtained from the literature. The probability distribution of dead loads is assumed to be normally distributed with a mean-to-nominal value ($\bar{G}/G_n$) of 1.05 and a COV of 0.1. The live loads follow an extreme Type I distribution with a mean-to-nominal value $\bar{Q}/Q_n$= 1.0 and a COV of 0.25 (Ellingwood, et al. 1980).

6. Simulation results

The basic idea of the methodology described in Section 2 is to establish an appropriate relationship between the reliability index and the system resistance factor for different steel frames. To achieve this, first the frames presented in Fig. 2 were designed based on the Australian standard (AS4100 1998). Member sizes of all frames as well as ultimate failure modes are given in Table 3. Those frames were then modeled into ABAQUS and analyzed by advanced analysis to evaluate the ultimate load factors (λ_n) under factored gravity loads (Table 3). The frame ultimate strength (R_n) can be expressed as the product of the total applied load and the ultimate load factor (total load × λ_n). By substituting this into Eq. 7, the total load can be cancelled out from both sides of the equation and the system resistance factor (φ_s) for each single frame, may be determined as $\varphi_s = 1/\lambda_n$.

To plot β versus φ_s two different approaches are presented in this study.

6.1 Method 1-adjusting frame cross-sections

The first method is based on adjusting the cross-sections of steel frame members to develop different ultimate load factors (λ_n) and thereby different resistance factors (φ_s) for each frame. For example for Frame 5 designed based on (AS4100 1998), the failure mode is BFY-CPY (beam(s) fully yielded and column(s) partially yielded) and the corresponding resistance factor is 0.68 ($\varphi_s = 1/\lambda_n$). This frame was then designed for different values of φ_s (system-based design) and the details of the cross-sections are presented in Table 4. Monte Carlo simulations are then conducted for these frames, accounting for all uncertainty as discussed in Section 4. The statistics (frequency distribution, mean and COV) of the ultimate load factors (λ) are determined and summarized in Table 5. An example of the frame ultimate load factor histogram, for the frame assigned to φ_s equals to 0.63, is shown in Fig. 5. A lognormal distribution is fitted to frame strength histogram. It should be noted that the mean and nominal values presented in Table 5 are based on applying unfactored nominal loads to the frame structures ($G_n + Q_n$).

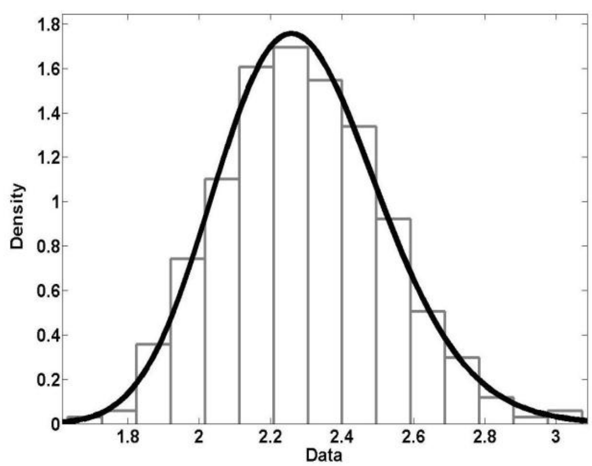

Figure 5: Histograms of ultimate load factor for Frame 5 with φ_s=0.63

Table 3: Frames design based on AS4100

Frame	Members	Section	λ_n	Failure mode
Frame 1	C_1, C_2 B_1	200UC59 460 UB74	1.32	BFY-CPY
Frame 2	C_1 to C_6 B_1, B_2 B_3	250UB37 310UB40 250UB31	1.39	BFY-CPY
Frame 3	C_1, C_3 C_2 B_1 B_2	460UB74 460UB67 410UB59 530UB92	1.63	BFY-CPY
Frame 4	C_1, C_3, C_4, C_6 C_2, C_5 C_7, C_8, C_9 B_1 to B_4 B_5, B_6	200UB25 250UB37 200UB22 360UB56 250UB37	1.26	CFY
Frame 5	C_1, C_4, C_5, C_8, C_9, C_{12} C_2, C_3, C_6, C_7 C_{10}, C_{11} B_1, B_3, B_4, B_6, B_7, B_9 B_2, B_5, B_8	250UC72 200UC59 150UC30 460UB67 360UB50	1.45	BFY-CPY
Frame 6	C_1 to C_8 C_9, C_{10} C_{11}, C_{12} B_1 to B_6 B_7 B_8	250UB37 250UB31 200UB29 360UB56 310UB40 200UB29	1.28	BFY-CPY
Frame 7	C_1, C_4 C_2 C_3, C_5, C_6 B_1, B_4 B_2 B_3	150UB14 310UB40 250UB37 310UB46 460UB67 200UB29	1.55	BFY-CPY
Frame 8	C_1, C_5, C_6 C_2, C_3 C_4 C_7, C_8 B_1 B_2 B_3, B_6 B_4, B_5	150UB14 250UB37 180UB18 200UB22 310UB40 360UB50 200UB29 250UB37	1.69	BFY-CPY
Frame 9	C_1, C_4, C_5, C_8, C_9 C_2, C_3, C_6, C_7 B_1 B_2, B_4 B_3, B_5 B_6	530UB92 200UB25 310UB46 250UB37 410UB53 610UB125	1.43	BPY-CPY

Using first order reliability analysis (FORM), the reliability index corresponding to any value of φ_s was determined, assuming different load ratios (Q_n/G_n) (Eq. 8). The system reliability index (β) versus φ_s for Frame 5 is plotted in Fig. 6. The appropriate system resistance factors are obtained for four values of target reliability, i.e. β= 2.5, 2.75, 3 and 3.5 (Table 6). To find a single resistance factor for each frame which does not depend on the specific load ratio, a relative weight is assigned to different load ratios (w_i). These weights represent the best estimate for the likelihood of different load situations (Ellingwood et.al 1980). Thus, the final system resistance factors were calculated based on Eq. 9 and presented in Table 6.

$$\varphi_s = (\sum w_i \times \varphi_{si})/100 \tag{9}$$

Table 4: System based design for Frame 5 fails by BFY-CPY

φ_s	Members	Section	λ_n
0.63	$C_1, C_4, C_5, C_8, C_9, C_{12}$	250UC72	1.4
	C_2, C_3, C_6, C_7	200UC59	
	C_{10}, C_{11}	150UC30	
	$B_1, B_3, B_4, B_6, B_7, B_9$	460UB74	
	B_2, B_5, B_8	360UB56	
0.74	$C_1, C_4, C_5, C_8, C_9, C_{12}$	200UC59	1.36
	C_2, C_3, C_6, C_7	200UC59	
	C_{10}, C_{11}	150UC30	
	$B_1, B_3, B_4, B_6, B_7, B_9$	460UB67	
	B_2, B_5, B_8	360UB50	
0.85	$C_1, C_4, C_5, C_8, C_9, C_{12}$	250UC72	1.18
	C_2, C_3, C_6, C_7	200UC59	
	C_{10}, C_{11}	150UC30	
	$B_1, B_3, B_4, B_6, B_7, B_9$	360UB56	
	B_2, B_5, B_8	360UB50	
0.96	$C_1, C_4, C_5, C_8, C_9, C_{12}$	250UC72	1.04
	C_2, C_3, C_6, C_7	200UC59	
	C_{10}, C_{11}	150UC30	
	$B_1, B_3, B_4, B_6, B_7, B_9$	360UB50	
	B_2, B_5, B_8	310UB40	

Table 5: Frame strength statistics, Frame 5

φ_s	Mean ($\bar{\lambda}$)	COV	λ_n	$\bar{\lambda}/\lambda_n$
0.63	2.292	0.100	2.143	1.07
0.68	2.089	0.101	1.959	1.07
0.74	1.959	0.100	1.832	1.07
0.85	1.692	0.102	1.593	1.06
0.96	1.483	0.100	1.399	1.06

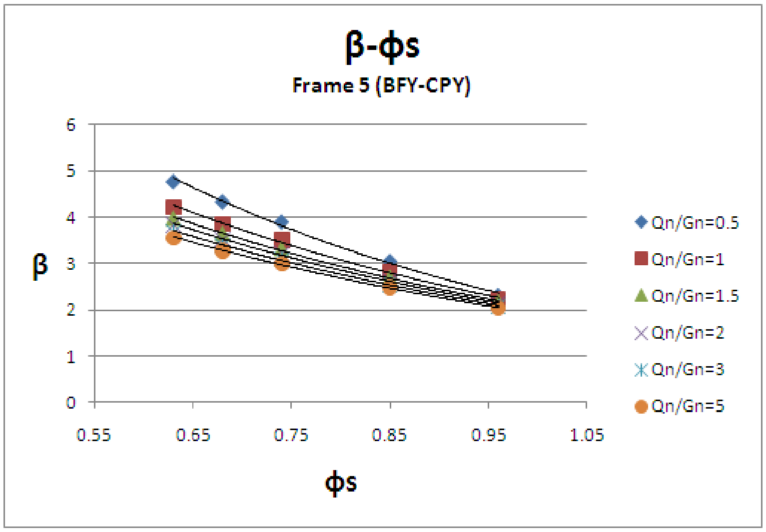

Figure 6: β vs. φ_s for Frame 5, using Method 1

Table 6: System resistance factor (φ_s) for Frame 5 for different reliability levels, Method 1

Q_n/G_n	Weight w (%)	φ_s			
		$\beta = 2.5$	$\beta = 2.75$	$\beta = 3$	$\beta = 3.5$
0.5	10	0.93	0.89	0.85	0.78
1	20	0.91	0.86	0.81	0.73
1.5	25	0.89	0.84	0.79	0.7
2	35	0.87	0.82	0.77	0.68
3	7	0.86	0.8	0.75	0.66
5	3	0.84	0.78	0.73	0.64
Final value of φ_s		0.89	0.84	0.79	0.71

6.2 Method 2 -adjusting applied loads

In second approach different values of φ_s for any specific frame are achieved by scaling the total applied load while the member cross-sections remain unchanged. Clearly, the mean-to-nominal values of frame strength $(\bar{R}/R_n)$ are same for frames assigned to different φ_s's. As only the ratio between the mean and nominal strength is reflected in design equation (Eq. 8), there is no need to run the simulations for every point in $\beta - \varphi_s$ plot. The Monte Carlo simulation is run for one frame with specific failure mode and the statistics of bias $(\bar{R}/R_n)$ are obtained. Reliability indices (β) are then determined, assuming different values of φ_s in Eq. 8. This method is more efficient as it is faster and needs fewer simulations to plot the $\beta - \varphi_s$ curves compared to Method 1. Fig .7 shows the reliability index versus the system resistance factor for Frame 5 fails by BFY-CPY and the final values of φ_s obtained for different target reliability are summarized in Table 7.

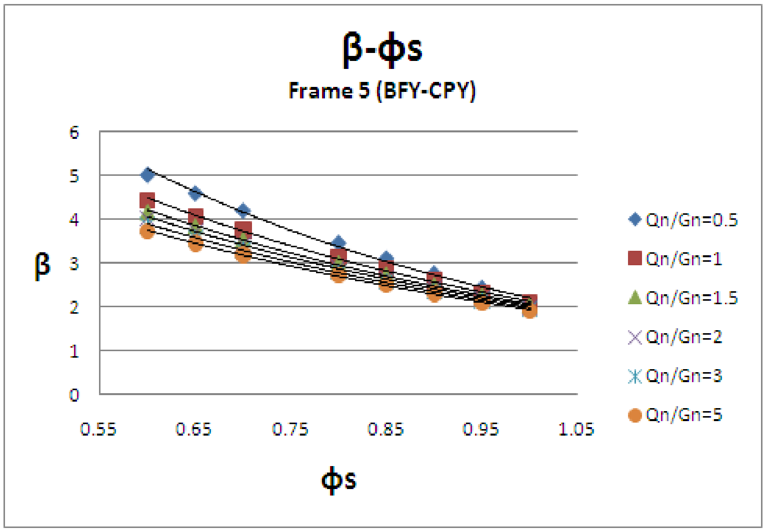

Figure 7: β vs. φ_s for Frame 5, using Method 2

Table 7: System resistance factor (φ_s) for Frame 5 for different reliability levels, Method 2

Q_n/G_n	Weight w (%)	φ_s			
		$\beta = 2.5$	$\beta = 2.75$	$\beta = 3$	$\beta = 3.5$
0.5	10	0.94	0.90	0.86	0.78
1	20	0.91	0.86	0.82	0.73
1.5	25	0.89	0.84	0.79	0.71
2	35	0.88	0.83	0.78	0.69
3	7	0.86	0.81	0.76	0.66
5	3	0.85	0.79	0.74	0.64
Final value of φ_s		0.89	0.84	0.79	0.71

The conclusion drawn from the results displayed in Table 6 and Table 7 is that both methods produce the same outcome for system resistance factors assuming various target reliability levels. Therefore, as Method 2 requires fewer simulations, this method is used in this study.

6.3 Simulation results for literature frames

In addition to the frames reported herein, a total of eight frames designed by Ziemian (1990) using advanced analysis are considered in this study to investigate the relationship between frame resistance factors and system reliability indices. These frames were previously analysed by Buonopane and Schafer (2006) and the values of φ_s for both β=2.0 and 3.0 were reported. The geometry, support conditions and applied loads for those frames are shown in Fig. 8 (a) while more necessary details are available in Buonopane and Schafer (2006). Two different cases of light and heavy gravity loads are considered. In present study, columns are subdivided into 8 elements and beams into sixteen elements using 2D beam elements of ABAQUS. All the random variables discussed in Section 4 are considered in frames simulations.

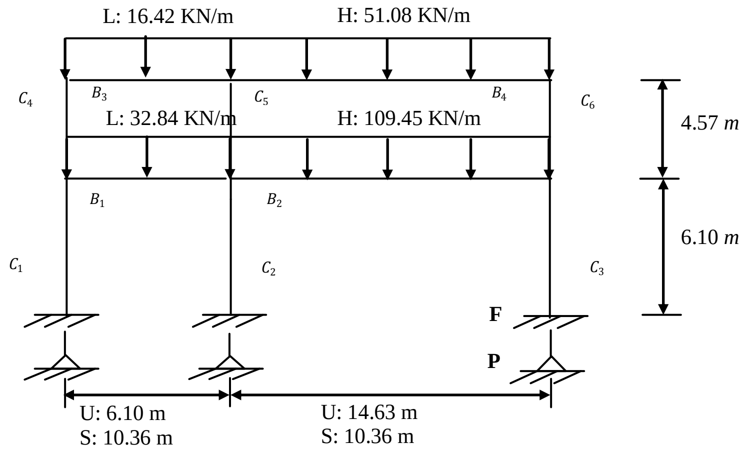

Figure 8: (a) Loads and dimensions of frames (Ziemian 1990)

The statistics of frame ultimate strength (R) are presented in Table 8. Normally distributed gravity load (W) with the mean of $1.026Q_n$ and the COV of 0.1 is assumed by Buonopane and Schafer (2006), which is generated from the normal distribution of dead load (mean=$1.05G_n$ and COV=0.1) and the normal distribution of live load (mean=Q_n and COV=0.1). A COV value of 0.1 seems to be suitable for dead load, but appears to be too low for live load. In this study, the COV of live load is updated to 0.25 as it is reported in most research studies (Ellingwood et.al 1980; Beck and Doria 2008). More accurately, instead of combining dead and live loads to generate the total load (W), the limit state of $R - G - Q$ is assumed and the corresponding reliability indices (β) and system resistance factors (φ_s) according to Section 5 are determined (Table 8). To design these frames (Ziemian 1990), resistance factors were incorporated into model by scaling the yield surface by the factor of 0.9. The resistance factors (φ_s) presented in Table 8, which are mostly about 0.9, verify the model.

Table 8: Statistic characteristics for Ziemian's frames

Frame	Mean ($\bar{R}$)	Bias ($\bar{R}/R_n$)	COV	Reliability index (β) using FORM and limit state of $R - G - Q$	φ_s
UP50HA	1.657	1.047	0.094	2.48	0.90
UP50LA	1.714	1.037	0.099	2.65	0.89
UF50HA	1.778	1.080	0.100	2.62	0.89
UF50LA	1.831	1.072	0.102	2.77	0.84
SP50HA	1.769	1.075	0.085	2.73	0.89
SP50LA	1.772	1.064	0.098	2.69	0.89
SF50HA	1.886	1.073	0.098	2.93	0.85
SF50LA	1.771	1.070	0.100	2.63	0.89

By applying Method 2 with frame strength statistics provided in Table 8, the system resistance factors (φ_s) are determined for all Ziemian's frames assuming different values of reliability indices (β) (Table 9).

Table 9: System resistance factor (φ_s) for frames designed by Ziemian (1990)

Frame	Failure modes	φ_s			
		$\beta = 2.5$	$\beta = 2.75$	$\beta = 3$	$\beta = 3.5$
UP50HA	BFY-CPY	0.88	0.83	0.78	0.70
UP50LA	BFY-CPY	0.86	0.81	0.77	0.69
UF50HA	BFY-CPY	0.89	0.84	0.79	0.70
UF50LA	BFY-CPY	0.89	0.84	0.79	0.71
SP50HA	BFY-CPY	0.92	0.86	0.81	0.72
SP50LA	BFY-CPY	0.89	0.84	0.79	0.71
SF50HA	BFY-CFY	0.90	0.85	0.80	0.71
SF50LA	BFY-CPY	0.89	0.84	0.79	0.71
Average value of φ_s		0.89	0.84	0.79	0.71

The mean value of system resistance factors (φ_s) for Zeimain's frames corresponding to target reliability of 3 is equal to 0.84 which is less than the mean value of 0.88 reported by Buonopane and Schafer (2006) for the same frames. The incorporation of more random variables (e.g. residual stress, elastic modulus and initial geometric imperfections) in this study leads to larger values of COVs and therefore smaller φ_s's for the same target reliability. Another reason can explain this difference is that Eq. 10 which is used in that study to determine the value of φ_s, might be an overestimate prediction of system resistance factors.

$$\varphi_s = (\bar{R}/R_n)\exp(-0.55\beta\, V_R) \tag{10}$$

6.4 Simulation results for proposed frames

A total of nine steel frames are studied in this paper to determine the system resistance factors using Method 2 (Fig. 2). Frames statistic characteristics as well as final values of φ_s are summarized in Table 10, assuming various reliability levels. Different failure modes are considered for each frame. The COVs of the strengths are in range of 0.093 to 0.106 and the mean-to-nominal strength ratio falls within the range of 1.02-1.12. Three different goodness-of-fit tests (Chi-Squared, Anderson Darling and Kolmogorov Smirnov (K-S)) are performed on ultimate strength results from simulations and log-normal distribution was found to be the best fit (Haldar and Mahadevan 2000).

Based on the results displayed in Table 10, it can be seen that for common type of rigid moment frames under gravity load, the system resistance factors obtained from extensive probabilistic study, are quite similar for specific target reliability, despite the frame configuration (regular or irregular) and frame failure modes (e.g. BFY-CPY, CFY,…). The values of φ_s range from 0.85 to 0.94 for target reliability of 2.5 with the average of 0.90 which somehow complies with the load factor of 0.9 for LRFD specification. As the target reliability increases to values of 2.75, 3 and 3.5, the average system reliability factor drops to 0.85, 0.8 and 0.7, respectively. Fig. 9 shows system resistance factor versus reliability index plots for some selected frames. Apparently, further work is needed to establish appropriate target reliability for system-based design by analyzing the reliability of the frames designed based on existing criteria.

Table 10: System resistance factor (φ_s), Bias factors and COVs of proposed frames

Frame	Failure mode	$\bar{R}/R_n$	COV	φ_s			
				$\beta = 2.5$	$\beta = 2.75$	$\beta = 3$	$\beta = 3.5$
Frame 1	BFY-CPY	1.07	0.103	0.89	0.83	0.79	0.71
Frame 2	BFY-CPY	1.03	0.100	0.85	0.80	0.76	0.68
	Instability	1.10	0.097	0.91	0.86	0.81	0.73
	BPY-CPY	1.10	0.093	0.92	0.87	0.82	0.73
Frame 3	BFY-CPY	1.07	0.102	0.88	0.83	0.79	0.71
	BPY-CFY	1.06	0.104	0.88	0.83	0.78	0.69
	BPY-CPY	1.02	0.105	0.85	0.80	0.75	0.67
Frame 4	BFY-CPY	1.07	0.102	0.91	0.85	0.79	0.69
	CFY	1.07	0.103	0.89	0.84	0.79	0.71
	BPY-CFY	1.07	0.105	0.89	0.84	0.79	0.71
Frame 5	BFY-CPY	1.07	0.101	0.89	0.84	0.79	0.71
	CFY	1.12	0.102	0.94	0.88	0.83	0.74
	BPY-CFY	1.11	0.101	0.90	0.85	0.81	0.73
Frame 6	BFY-CPY	1.06	0.101	0.87	0.82	0.78	0.71
	BPY-CFY	1.08	0.104	0.89	0.84	0.79	0.71
Frame 7	BFY-CPY	1.06	0.099	0.88	0.83	0.79	0.71
	CFY	1.04	0.095	0.88	0.82	0.79	0.70
	BPY-CPY	1.04	0.095	0.88	0.82	0.77	0.69
Frame 8	BFY-CPY	1.06	0.102	0.88	0.83	0.78	0.70
	BPY-CFY	1.07	0.102	0.89	0.84	0.79	0.71
	BPY-CPY	1.06	0.101	0.90	0.84	0.79	0.70
Frame 9	BFY-CPY	1.06	0.106	0.87	0.82	0.78	0.70
	BPY-CPY	1.07	0.103	0.91	0.85	0.80	0.71
	BFY-CFY	1.09	0.103	0.90	0.84	0.79	0.71
	Average value of φ_s			0.90	0.85	0.80	0.70

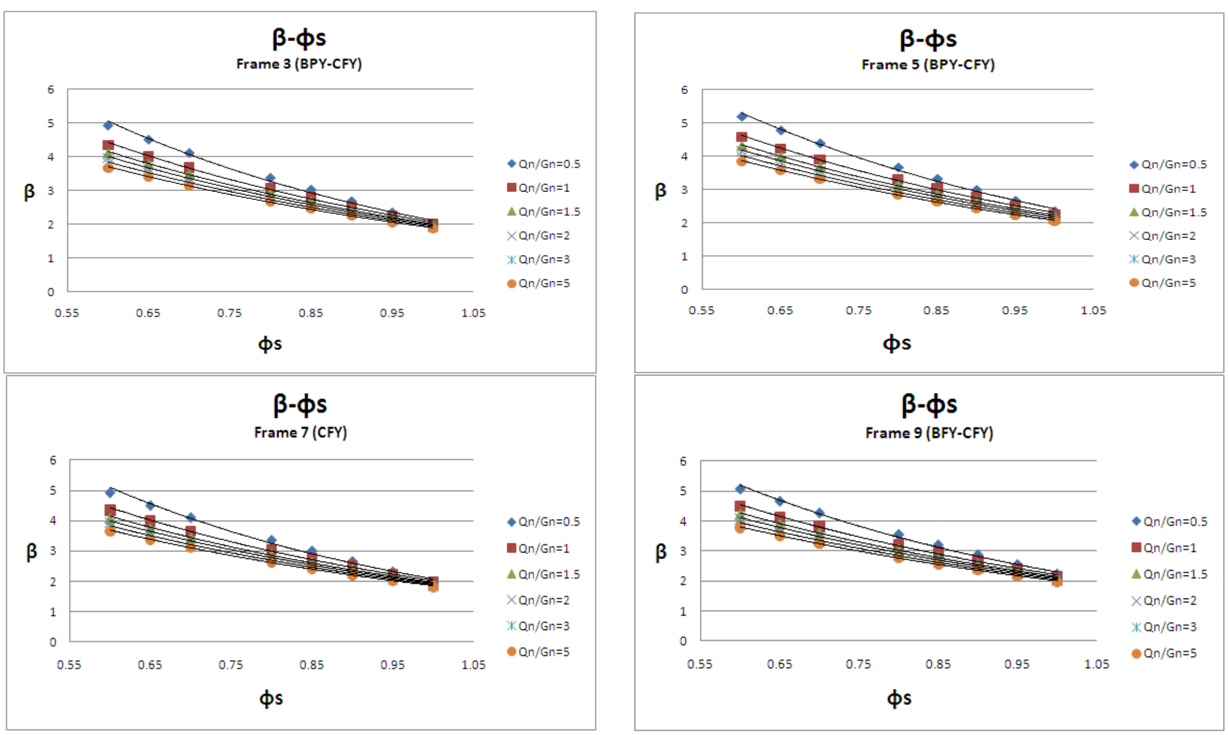

Figure 9: β vs. φ_s for selected frames, using Method 2

7. Conclusion

This paper is part of the research effort to develop the next generation of steel structural code based on advanced analysis. Using that approach and derived system resistance factor, the user stands to benefit from more reliable design method and shortened design time as there is no need for separate member/section capacity check. This method can provide the details of frame failure modes. In most cases using system-based design by advanced analysis can save portion of steel weight and leads to lighter structures.

To develop this methodology, a series of 2D frames were analyzed in this study using advanced analysis and Latin Hypercube sampling method. The effect of uncertainties in ultimate strength of a frame is considered by modeling yield stress, elastic modulus, cross-sectional properties, member and frame initial geometric imperfection and residual stress as random variables. Different failure modes are considered for each frame the system strength statistical characteristics are obtained. The resistance factor φ_s is plotted versus reliability index (β) for all frames using two different approaches. It was concluded that Method 2, which is based on changing loads to achieve a specific resistance factor, is faster and requires fewer simulations. The simulation results show that although different frames with various geometries and configurations are analyzed, the COVs and mean-to-nominal ratios of ultimate strength are quite similar. On the other hand different frames fail by various failure modes, show nearly same statistics and thereby similar values of system resistance factors (φ_s). The system resistance factor determined in this study for target reliability of 2.5 is 0.9 while it drops to 0.7 by assuming the reliability of 3.5. However, these values are only based on the result of simulations for unbraced frames under gravity and further work is needed to determine φ_s for braced frame as well as frames under different load combinations.

Acknowledgements

The authors would like to express their gratitude to Prof. Bruce R. Ellingwood, from the University of Georgia Tech, for his insightful comments on this study that form the basis of this paper.

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