RELIABILITY OF STEEL FRAME SYSTEMS WITH SEMI-RIGID CONNECTIONS

by H. P. Hong and S. Wang

Department of Civil and Environmental Engineering
University of Western Ontario, London, Canada, N6A 5B9

September 2003

ICLR Research
Paper Series – No. 35

ISBN 0-9733795-2-9

ABSTRACT

Often the connections of steel frames are assumed to be fully rigid or ideally pinned when evaluate the system reliability of steel frames, although it is well known that such idealization may not be adequate since the connections are better modeled as semi-rigid connections. This study is focused on evaluating the reliability of steel frame systems with rigid connections and with semirigid connections. The system reliability of frames is assessed using simulation technique. The limit state function for the system reliability analysis is established based on the collapse load factor which is obtained using the second-order refined plastic-hinge analysis method. The uncertainty considered are the loads and material yield strength. The results indicate that although the reliability of frame systems with rigid connections is always higher than that of frame systems with semirigid connections, however, from reliability point of view, in most cases their differences are not very significant. Therefore, it appears that if the quality control of the construction of the connections is adequate, the consideration of whether the connections are rigid or semirigid does not significantly impact the system reliability, although it could affect the estimated expected cost of the system which includes the initial cost and cost of collapse.

Key words: Probability of failure, reliability index, system, nonlinear, inelastic.

1.0 Introduction

In the literature, often the reliability analysis of the steel frames is carried out by assuming that the connections between the beams and the columns are either fully rigid or ideally pinned. The fully rigid connection assumption implied that full slope continuity exists between the adjoining members and that the full gravity moment is transferred from the beams to the columns. On the other hand, the ideally pinned connection assumption implies that there is no restraint for rotation between the beams and the columns and the columns carry no gravity moment transferred from the beams. These assumptions simplify the structural analysis but may not mimic the observed structural behaviour. It has been indicated (e.g., Chen et al. 1996) that stiffness of the current practical connections falls between that of fully rigid and that of ideally pinned. In other words, the connections are semirigid connections. The semirigid connections may affect the frame structural behaviour and consequently the estimated system reliability.

It should be noted that structures including the steel frame structures are designed using the load and resistance factors given in design standards. These factors are calibrated such that structural members designed according to codes meet, on average, a set of pre-selected target reliability levels (Ellingwood et al. 1980). Therefore, it is expected that a well-designed structural system is at least as safe as the most critically loaded structural element since the system reliability is always larger than or equal to the element reliability. However, quantitative assessment of the system reliability is much more difficult than that of a structural element. System reliability evaluation of steel frame structures has been reported in the literature (Kam et al. 1983; Thoft-Christensen and Murotsu 1986; Bennett and Ang 1987; Zimmerman et al. 1992, Haldar and Zhou 1992; Zhao and Ono 1998; Zhou and Hong 2000). The approaches taken by these studies can be divided into two categories. The first one is the failure modes based approach and the second one is the collapse load factor based approach. The former requires identifying all the dominant failure modes of the structure. Unfortunately, efficiently and robustly identifying all the dominant failure modes generally presents considerable difficulty. Moreover, the computation of overall failure probability contributed from the dominant failure modes is also a formidable task mainly due to the correlation among the failure modes. The latter is to use a limit state function of the system established directly from the so-called collapse load factor of the system (Kam et al. 1983; Haldar and Zhou 1992; Zhao and Ono 1997). The collapse load factor based approach is further extended recently for nonproportional loading

(Zhou and Hong 2000). They showed that for typical low-rise industrial buildings the system reliability is much higher than the reliability of the most critically loaded structural member due to the beneficial effects of force redistribution. The ratio between the probability of failure of the most critical member and of the system ranges from 6 to 20 and depends on the structural configuration, and the degree of redundancy.

In this study, the collapse load factor based approach is adopted for the reliability evaluation of the steel frame systems with rigid and semirigid connections. For the numerical evaluation simple Monte Carlo technique is employed. The estimated reliabilities for frames with semirigid connections are compared to those of identical frames but with rigid connections. Detailed analysis procedure and results are presented in the following sections.

2.0 Limit state function and analysis procedure

2.1 Limit state function

Let **P** and **R** denote the vectors of random variables that represent the external loads and the resistance of the structure, respectively. **P** may include the dead load, live load, and environmental loads while **R** may include the yield strength and modulus of elasticity of steel, cross-sectional properties of structural members, and the geometry of the structure. Further let **p** and **r** denote values of **P** and **R**, respectively. For a given structure with resistance **r** subjected to loads **p** that are applied proportionally, the load carrying capacity of the structure can be expressed as $\lambda\mathbf{p}$, where λ that depends on **p** and **r** is known as the load factor or the collapse load factor in the plastic analysis. Therefore, $\lambda > 1.0$ indicates that the structure can withstand load **p** while $\lambda \le 1.0$ indicates that the structure will collapse under **p**. The values of **P** and **R** leading to the collapse can be expressed as $g_s(\mathbf{r},\mathbf{p}) \le 0$, where

$$g_s(\mathbf{r},\mathbf{p}) = \lambda(\mathbf{r},\mathbf{p}) - 1 \tag{1}$$

$g_s(\bullet)$ represents the limit state function, and $\lambda = \lambda(\mathbf{r},\mathbf{p})$ is used to emphasize that λ is a function of **r** and **p**.

The collapse load factor $\lambda(\mathbf{r},\mathbf{p})$ can be evaluated by one of the many second-order inelastic frame analysis methods which take into account the interaction between the axial load and bending moment, initial imperfections, the geometric nonlinearity (second-order effects), and the distributed plasticity (Chen et al. 1996). A plastic-zone analysis that includes distributed yielding effects, residual stresses, initial geometric imperfections, and many other significant behavioral effects will certainly be the most refined and accurate one. However, this model is too computationally intensive to be employed in the probabilistic analysis. A second-order elastic-plastic hinge analysis that employs concentrated plastic hinges is computationally efficient. However, it can lead to significantly unconservative errors (Liew 1992). To overcome the inaccuracy of the plastic-hinge analysis, Liew (1992, see also Chen et al. 1996) proposed a second-order refined plastic-hinge model which uses a column tangent-modulus to represent the distributed yielding due to axial-force effects and a plastic-hinge stiffness-degradation model to represent the distributed yielding due to flexure. They implemented this model in a program called PHINGE and obtained numerical results suggesting that the model provides sufficiently accurate predictions for a wide range of structures. Therefore, by considering both accuracy and numerical efficiency, the refined plastic-hinge model is adopted in this study for evaluating $\lambda(\mathbf{r},\mathbf{p})$.

If the semirigid connection is considered, the rigidities of the connections represented by the slope of the connection moment M and the connection relative rotation θ_r given by Chen et al. (1996) in the following is adopted,

$$M = \frac{\theta_r / \left(M_u / R_{ki}\right)}{\left(1+\left(\theta_r / \left(M_u / R_{ki}\right)\right)^n\right)^{1/n}} M_u \tag{2}$$

In the above equation, M_u is the ultimate moment capacity of the connection, R_{ki} is the initial connection stiffness and n is a shape parameter.

2.2 Analysis procedure

Based on the established limit state equation shown in Eq. (1), the probability of failure of the structure, P_{fs}, can be expressed as

$$P_{fs} = \int_{g_s<0} f_{\mathbf{R}}(\mathbf{r})\, f_{\mathbf{P}}(\mathbf{p})\, d\mathbf{r}\, d\mathbf{p} \tag{3}$$

where $f_{\mathbf{R}}(\mathbf{r})$ is the joint probability distribution function of the resistance, $f_{\mathbf{P}}(\mathbf{p})$ = joint probability distribution function of the loads.

The integral equation shown in Eq. (3) may be evaluated using the first order reliability method (FORM) (Madsen et al. 1986) and simple simulation technique. The FORM is very efficient if $g_s(\mathbf{r},\mathbf{p})$ is smooth such that its first order derivatives exist. Since the failure of the system may be contributed from different failure modes, the use of the FORM may lead to error because its use for the limit state surface with multiple local minimum points may not be adequate. The simple simulation technique is less computationally efficient than the FORM, and its accuracy is not affected by multiple failure points on the limit state surface. Furthermore, the simulation is straightforward for implementation, and numerical difficulties are not likely to occur during the analysis. Therefore, this method is employed for the numerical analysis results presented in the next section.

3.0 Numerical results

For the numerical analysis, four steel frames shown in Figures 1 to 4 (Chen et al. 1996), two with rigid connections and two with semirigid connections, are considered. Two of the frames are unbraced and the other two are braced. Both pinned base and rigid base are considered.

The nominal steel yield strength is 248 (MPa). The design yield strength is taken as 0.9 times the nominal yield strength. Frames are subjected to the live load and dead load. All beams are modeled by four discrete elements in order to detect the possible formation of plastic hinges within a beam and, all columns by one element due to the absence of transverse loads. The gravity loads are applied as point loads at the beam quarter points. The loads are applied at the nodal points in 5% increments with respect to the full factored loads. The dimensions and the loads are also shown in Figures 1-4. The loads shown on the figures are the factored loads which include both dead and live loads. The nominal dead to live load ratio is considered to be equal to one. The dead load factor of 1.20 and the live load factor of 1.60 are employed to calculate the nominal dead and live loads.

For the reliability analysis, dead load and live load are considered. These loads and the steel yield strength are considered random. The statistics of the random variables used for the reliability analysis are shown in Table 1. These statistics are in agreement with those used by

Galambos and Ravindra (1978) and Ellingwood et al. (1980) for design code calibration studies. The random variables are assumed to be independent of each other and the loads on different floor levels are fully correlated to each other. For the numerical evaluation, geometrical uncertainties are neglected. Further, the uncertainty associated with stiffness of semirigid connection is ignored. The values of the parameters for the stiffness of the semirigid connections (see Eq. (2)) employed are M_u = 200.4 (kNm), R_{ki} =107832.1 (kNm/rad) and n = 0.8.

The system reliabilities obtained using the procedure outlined in previous section with a simulation cycle of 50000 are shown in Table 2 for the frames. The table also shown the corresponding reliability index β which is commonly considered for the design code calibration studies. This index β is obtained from $\Phi^{-1}(P_{fs})$, where $\Phi^{-1}(\bullet)$ represents the inverse of standard normal distribution function.

The results shown in Table 2 suggest that the system reliability of the semi-rigid frames is less than that of the rigid frames with identical element. Comparison of the results for the frames with fixed base and for the frames with pinned base suggest that under vertical loads the condition of the support is not very significant, especially, if the frames are unbraced. The probability of failure of the braced frames is smaller than that of unbraced frame. This is expected since no lateral loads are considered and the bracing is usually employed for system stability and for controlling the lateral drift. In almost all cases, the failure probabilities of frames with semi-rigid connections appeared to be about 3 times less than those of frames with semirigid connections. This difference in probability of failure which is less than an order of magnitude may be considered to be not very significant since the use of the load and resistance factors suggested in design codes may leads to the designed structural members having such a different failure probability level.

4.0 Discussion and conclusions

An procedure for evaluating the system reliability of steel frame systems is outlined and employed for numerical analysis. The approach based on the collapse load factor is taken to establish the limit state function. The analysis results for four pairs of steel frame structures showed that ignoring the connection flexibility can be unconservative. In almost all cases, the failure probabilities of frames with semi-rigid connections appeared to be about 3 times less than those of frames with semirigid connections. Form reliability point of view, this difference in probability of failure which is less than an order of magnitude may be considered to be not very significant since the use of the load and resistance factors suggested in design codes may leads to the designed structural members having such a different failure probability level. However, it could impact the estimated expected cost which includes the initial cost and cost of collapse of a structural system.

It must be emphasized that this study is focused on the effect of the connection flexibility on the system reliability of steel frame structures. Further, only vertical loads are considered and the possible uncertainty associated with semirigid connections is ignored. The conclusions reached should be verified for frames subjected to horizontal loading due to earthquake and wind loads and for frames with uncertain rigidity of flexible connections.

5.0 Acknowledgements

The financial support of ICLR is gratefully acknowledged.

REFERENCES

Bennett, R. M. and Ang, A. H.-S. (1987). "Formulation of Structural System Reliability." Journal of Engineering Mechanics, ASCE, 112(11), 1135-1151.

Chen, W. F., Goto, Y., and Liew, J. Y. R. (1996). *Stability Design of Semi-rigid Frames*. John Wiley & Sons, Inc., New York.

Ellingwood, B., Galambos, T. V., MacGregor, J. G., and Cornell, C. A. (1980). "Development of a probability based load criterion for American national standard A58." Spec. Pub. No. 577, National Bureau of Standards, Washington, D.C.

Galambos, T. V. and Ravindra, M. K. (1978). "Properties of Steel for Use in LRFD." Journal of the Structural Division, ASCE, 104(9), 1459-1468.

Haldar, A. and Zhou, Y. (1991). "An Efficient SFEM Algorithm for Nonlinear Structures." *Computational Stochastic Mechanics*, Computational Mechanics Publications, Southampton, 851-862.

Kam, T. Y., Corotis, R. B., and Rossow, E. C. (1983). "Reliability of Nonlinear Framed Structures." Journal of Structural Engineering, ASCE, 109(7), 1585-1601.

Liew, J. Y. R. (1992). "Advanced Analysis of Frame Design." PhD Thesis, Purdue University, West Lafayette, IN.

Madsen, H. O., Krenk, S., and Lind, N. C. (1986). *Methods of Structural Safety*. Prentice-Hall, Englewood Cliffs, N.J.

Throft-Christensen, P. and Murotsu, Y. (1986). *Application of Structural Systems Reliability Theory*. Springer-Verlag, Berlin.

Zhao, Y. G. and Ono, T. (1997). "System Reliability Evaluation of Ductile Frame Structures." Journal of Structural Engineering, ASCE, 124(6), 678-685.

Zhou, W and Hong, H. P. (2000) "Reliability Analysis of Steel Frames." Proceedings of 28th Annual Conference of the Canadian Society for Civil Engineering, London, Ontario.

Zimmerman, J. J., Ellis, J. H., and Corotis, R. B. (1993). "Stochastic Optimization Models for Structural Reliability Analysis." Journal of Structural Engineering, ASCE, 119(1), 223-239.

List of Figures

Captions

Table 1. Statistics of the random variables

Random Variable	Mean/Nominal	Coefficient of Variation	Probability Distribution Type
Dead Load	1.0	0.08	Normal
Live Load	1.0	0.25	Gumbel
Steel Yield Strength	1.05	0.10	Lognormal

Table 2. Comparison between the semi-rigid frame and rigid frame with fixed base

Base	P_{fs} or β	Rigid frame (rigid connections)		Semi-rigid frame (Semirigid connections)	
		Unbraced	Braced	unbraced	Braced
Fixed base	P_{fs}	0.26×10^{-3}	$<0.2\times10^{-4}$	0.76×10^{-3}	0.74×10^{-3}
	β	3.46	>4.10	3.17	3.18
Pinned base	P_{fs}	0.29×10^{-3}	0.26×10^{-3}	0.75×10^{-3}	0.72×10^{-3}
	β	3.44	3.46	3.18	3.19

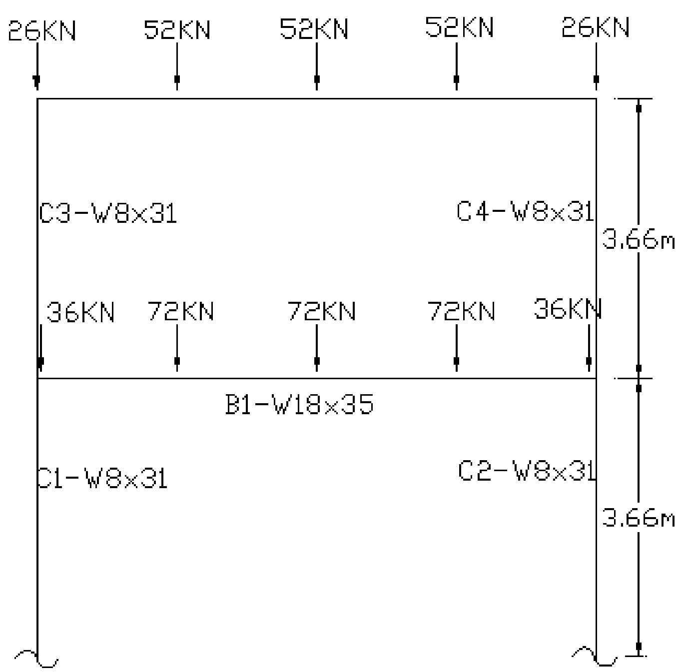

Figure 1. Hong and Wang.

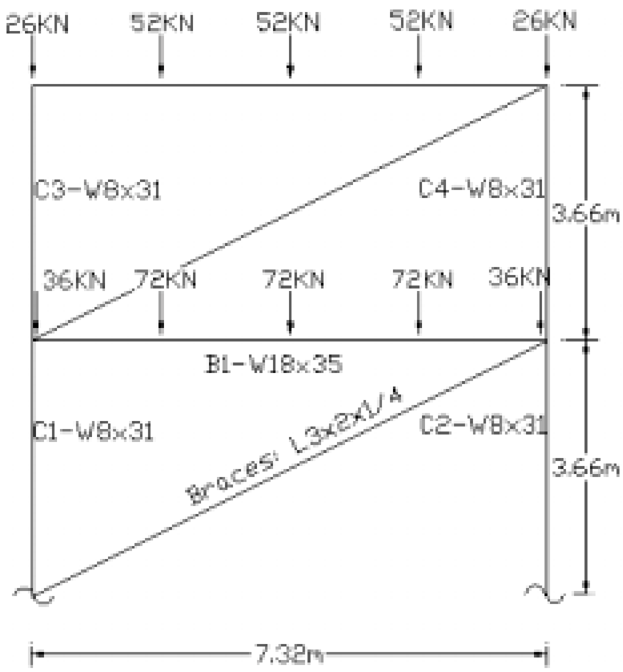

Figure 2. Hong and Wang.

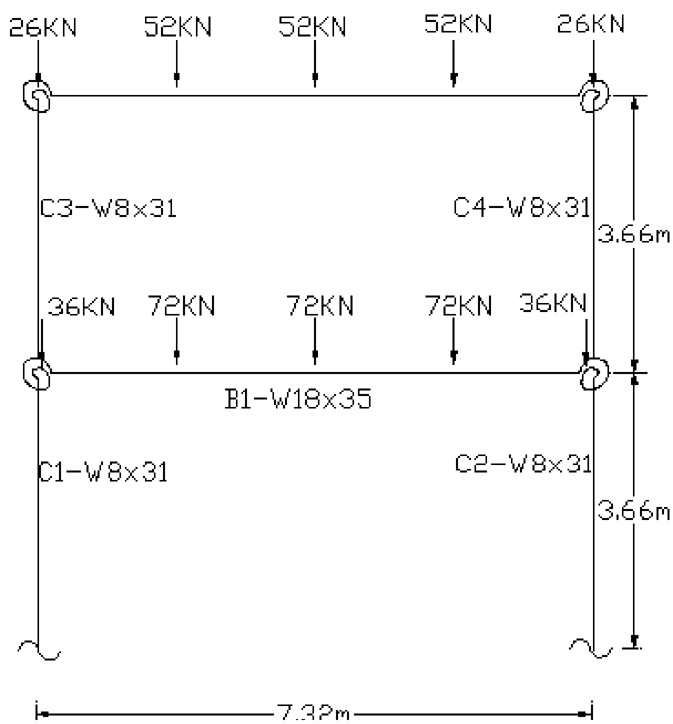

Figure 3. Hong and Wang.

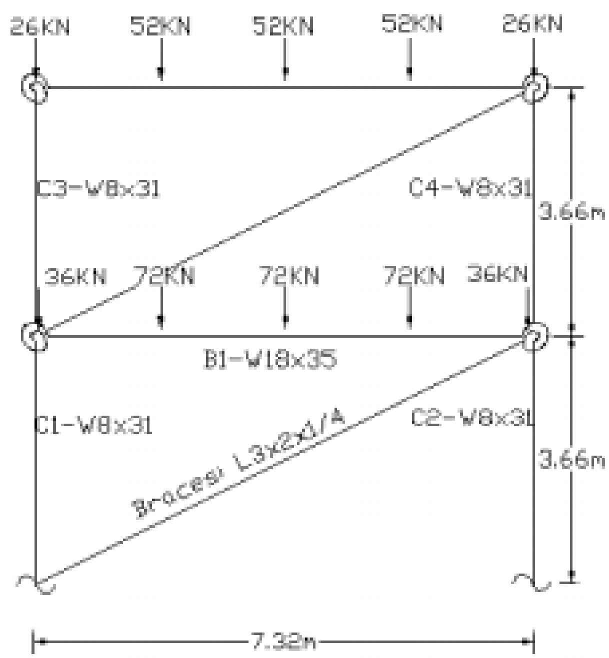

Figure 4. Hong and Wang.