강 뼈대 구조물의 지진위험도 평가에 대한 부분구속 접합부의 영향

Effect of Partially Restrained Connections on Seismic Risk Evaluation of Steel Frames

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요 지

본 논문은 강 뼈대 구조물의 보-기둥 접합부의 사실적인 상태를 묘사하기 위하여 부분구속 접합부(Partially Restrained (PR) connections)를 고려한 지진하중 상태하의 신뢰성 해석과 접합부 및 그들에 내재된 불확실성이 구조물의 위험도에 미치는 영향에 관한 연구이다. 신뢰성해석을 위하여 응답면기법(Response Surface Method), 유한요소법(Finite Element Method), 일차신뢰도법(First Order Reliability Method), 그리고 반복선형보간 기법(Iterative Linear Interpolation Scheme)을 효과적으로 결합한 추계유한요소법(Stochastic Finite Element Method)을 제안하였다. 일반적으로 모멘트-상대회전각 곡선에 의해서 표현되는 보-기둥에 대한 부분구속 접합부(PR connections)의 거동이 본 논문에서는 네 개의 매개변수를 사용하는 리차드 모델(Four-parameter Richard Model)을 사용하여 모사하였다. 지진하중에 대하여, 부분구속 접합부에서의 재하(Loading), 제하(Unloading) 및 재재하(Reloading) 거동은 모멘트-상대회전각 곡선과 Masing 법칙을 사용하여 표현하였다. 시간영역에서 지진가속도를 구조물에 작용시킴으로써 지진하중의 사실적인 재현을 도모하였다. 다양한 주요 비선형성의 원인들을 고려한 부분구속 접합부를 가지는 강 뼈대 구조물의 신뢰성해석이 지진위험도를 평가하기 위하여 수행되었다. 제안된 기법의 명확한 이해를 돕기 위하여 한 예제를 제시하였다.

핵심용어 : 부분구속접합부, 지진하중, 신뢰성, 한계상태함수, 응답면기법, 추계 유한요소법

Abstract

The effect of partially restrained(PR) connections and the uncertainties in them on the reliability of steel frames subjected to seismic loading is addressed. A stochastic finite element method(SFEM) is proposed combining the concepts of the response surface method(RSM), the finite element method(FEM), the first-order reliability method (FORM), and the iterative linear interpolation scheme. The behavior of PR connections is captured using moment-relative rotation curves, and is represented by the four-parameter Richard model. For seismic excitation, the loading, unloading, and reloading behavior at PR connections is modeled using moment-relative rotation curves and the Masing rule. The seismic loading is applied in the time domain for realistic representation. The reliability of steel frames in the presence of PR connections is calculated considering all major sources of nonlinearity. The algorithm is clarified with the help of an example.

Keywords : *partially restrained connections, seismic loading, reliability, limit state function, response surface method, stochastic finite element method*

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1. Introduction

Structural failures reported in the post-earthquake investigation of Northridge and Kobe earthquakes forced the research profession to reevaluate issues related to the seismic design and analysis of steel structures. Especially the connection of steel frame structures and their risk assessment associated with the extreme loading conditions such as earthquakes have received great attention from them. Therefore, extensive research works have been being conducted in this subject by numerous researchers from all over the world. However, an efficient and accurate reliability estimation method for nonlinear steel frame structures in consideration of realistic connection conditions subjected to seismic loading has not yet been proposed. Most structures develop nonlinear behavior just before failure. To evaluate the damage state of a structure just before failure, various sources of nonlinearity under dynamic loading conditions need to be considered. One of the major sources of nonlinearity in steel frames is the presence of partially restrained(PR) connections. Connections in steel frames are routinely considered to be fully restrained(FR). The FR connections are assumed to simplify calculations and are thus a major weakness in current analytical procedures. An analogous situation is assuming that truss members are pin-connected and develop only axial loads in order to simplify calculations. However, it is established in the profession that connections in steel frames are PR type with different rigidities. The implication is that PR connections change the dynamic properties, e.g., the natural frequency, stiffness, damping, etc., of structures. PR connections reduce the overall stiffness of frames but add a major source of energy dissipation and a type of hysteretic damping to the structure. The loading, unloading, and reloading behavior at PR connections needs to be captured appropriately to evaluate the reliability of steel frames. Consequently, for seismic response analysis, proper consideration of the rigidity of connections is essential no matter how difficult the analysis procedure becomes.

Huh[1] and Huh & Haldar[2] proposed a stochastic finite element method(SFEM) integrating the response surface method(RSM), the finite element method(FEM), the first-order reliability method (FORM), and the iterative linear interpolation scheme to estimate the reliability of structures subjected to short-duration dynamic loading including seismic loading. The presence of PR connections in a frame was not considered. The algorithm is extended in this paper to consider PR connections. The proposed algorithm considers all major sources of nonlinearity and uncertainty in evaluating the reliability of steel frame structures as realistically as possible.

2. Connection Conditions

Most structures consist of many structural elements, which are connected to each other by various types of connections. For steel structures, these connections are usually modeled as FR and PR types. However, extensive experimental studies indicate that they are essentially PR connections with different rigidities as explained in the previous section. In a deterministic analysis, PR connections add a major source of nonlinearity by decreasing the overall stiffness of a frame, and in the respect of dynamic analysis they add a major source of energy dissipation, *causing an additional major source of uncertainty* in reliability analysis. Thus, the reliability of steel frames with PR connections subjected to seismic excitation is expected to be quite different than that of frames with FR connections. The implications of the presence of PR connections and the uncertainty in modeling them in the analysis are the subjects of this paper. Some of essential issues on connections and their incorporation into the proposed algorithm are explicitly discussed in the following sections.

2.1. Classification of Flexible Connections

The flexibility of connections particularly in a steel frame should be properly considered in any realistic analysis algorithm. The best description of the flexural behavior of a connection is its moment-rotation M-θ curve. This is the relationship between the moment transmitted by the connection and the rotation of the beam relative to the column. The typical moment and rotation angle position is shown in Fig. 1.

The American Institute of Steel Construction (AISC) specifications[3] provide three basic types of connections and associated analysis assumptions:

1. Type I, designated as "rigid-frame"(continuous frame), assumes that beam-to-column connections have sufficient rigidity to maintain the original angles between intersecting members virtually unchanged.
2. Type II, designated as "simple-frame"(unrestrained or pinned), assumes that the end of the beams and girders are connected for shear only, and are free to rotate under gravity load.
3. Type III, designated as "semi-rigid"(partially restrained, PR), assumes that the connections of beams and girders possess a dependable and known moment capacity intermediate in degree between the rigidity of Type I and flexibility of Type II.

The Load and Resistance Factored Design (LRFD) code[4] also suggests two types of models for connections: fully restrained(FR) and partially restrained(PR). The graphical representation of these classifications is shown in Fig. 2.

However, despite these classification, it has been found from experimental observation[5),6),7)] that the connections are rarely rigid(Type I) or pinned(Type II) as is routinely assumed in analysis. They are partially restrained(Type III, PR) with different rigidities in most cases. The consideration of connection rigidity is essential in the analysis and design of large deformed steel frame structures[6)].

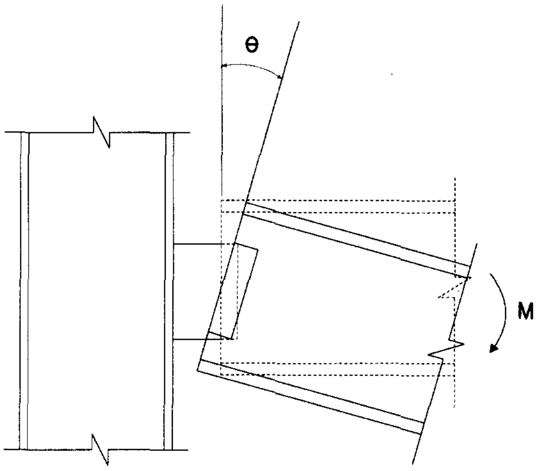

Fig. 1 Moment and relative rotation(M-θ) of a flexible connection

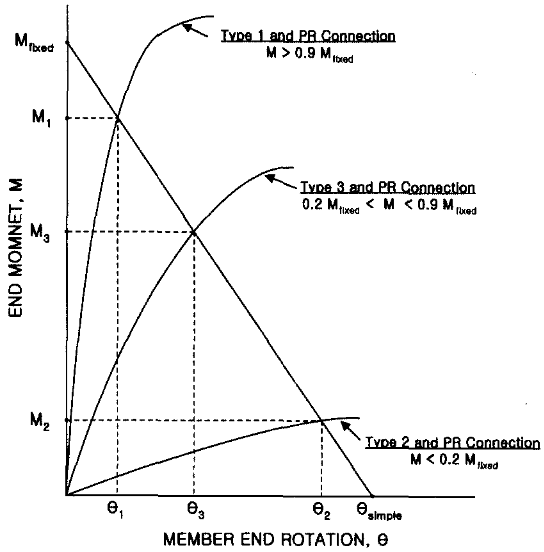

Fig. 2 M-θ Characteristics of AISC Connection Types

2.2. Moment-Rotation Curve Models

To study the effect of connection flexibility on the behavior of frames, each connection has to be modeled properly. Various models have been proposed by researchers to represent the moment-rotation behavior for connections, such as linear, bilinear, piecewise linear, polynomial, exponential, B-Spline, and Richard models. The linear model cannot be used since the M-θ curves for steel beam-to-column connections are in nature nonlinear. The bilinear and piecewise linear models are simple to use, they may, however, produce numerical difficulties due to abrupt changes in the stiffness. The polynomial model could produce negative connection stiffness unless the terms in the model are selected appropriately. The exponential and the B-Spline models usually give a good representation of the M-θ curve, but a large number of parameters need to be evaluated for their use.

The Richard model[8),9)] was developed using actual test data to predict the behavior of a wide variety of steel frame connections(36 composite and non-composite connections). The laboratory force-deformation test results are analytically described by the Richard four-parameter function. These parameters are dependent upon the connection geometry, stiffness and strength. A generalized analytical connection model that simulates the connection segments is then used to generate the moment-rotation curve. It has good predictability and is easy to incorporate into the general finite element formulation. Therefore, the Richard model is used to represent the M-θ curve in this study and is further explained in the following section.

2.3 Consideration of Connection Conditions in the Finite Element Formulation

The presence of partial connection rigidity needs to be incorporated in the deterministic analysis of structures to capture their realistic behavior. As mentioned in the previous sections, the relationship between the moment M, transmitted by the connection, and the relative rotation angle θ is generally used to represent the flexible behavior of a connection and the Richard four-parameter moment-rotation model is chosen here to represent the flexible behavior of a connection. This model can be expressed as[8)]:

$$M = \frac{(k-k_p)\theta}{\left(1+\left|\frac{(k-k_p)\theta}{M_0}\right|^N\right)^{1/N}} + k_p\theta \qquad (1)$$

where M is the connection moment, θ is the relative rotation angle between the connecting elements, k is the initial stiffness, k_p is the plastic stiffness, M_0 is the reference moment, and N is the curve shape parameter.

These parameters are identified in Fig. 3. This model also encompasses more simple models, that is, if k_p is k, it becomes the simple linear model, the elasto-plastic model if $k_p=0$ and the bilinear model if N is large.

The assumed stress-based FEM approach[10),11),12)] is used for the structural analysis in this study because of its efficiency in incorporating PR connections in the algorithm. In this approach,

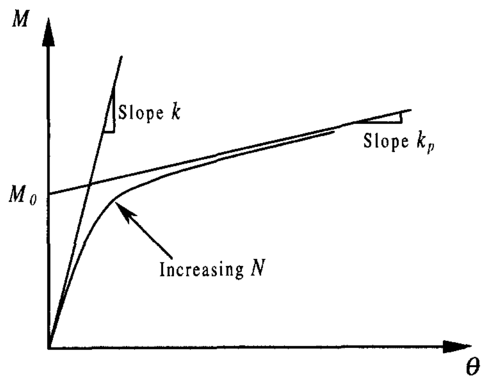

Fig. 3 M-θ curve using the Richard Model a beam-column element is introduced to model both a regular structural element and a flexible connection. Although an ordinary beam-column element is used to represent a PR connection element for numerical analyses, its stiffness should be updated at each iteration since the stiffness representing the partial rigidity depends on θ. This can be accomplished by updating the Young's modulus as:

$$E_C(\theta)=\frac{l_C}{I_C}K_C(\theta)=\frac{l_C}{I_C}\frac{\partial M(\theta)}{\partial\theta} \tag{2}$$

where l_C, I_C, and $K_C(\theta)$ are the length, the moment of inertia, and the tangent stiffness of the connection element, respectively. $K_C(\theta)$ is calculated using Equation (1) and can be shown to be:

$$K_C(\theta)=\frac{dM}{d\theta}=\frac{(k-k_p)}{\left(1+\left|\frac{(k-k_p)\theta}{M_0}\right|^N\right)^{\frac{N+1}{N}}}+k_p \tag{3}$$

The Richard model for PR connections discussed up to now represents only the monotonically increasing loading portions of the M-θ curves. However, the unloading and reloading behavior of the M-θ curves are also essential for nonlinear seismic analysis. Namely, if the direction of the moment to be applied in a connection is reversed, the connection is expected to unload and follow a different path, which is almost linear with a slope equal to the initial slope of the M-θ curve as shown in Fig. 4. This subject was extensively addressed in the literature[13),14)]. To consider the unloading and reloading behavior at the PR connections, the monotonic loading behavior and the Masing rule are used. A general class of Masing models can be defined with a virgin loading curve expressed as:

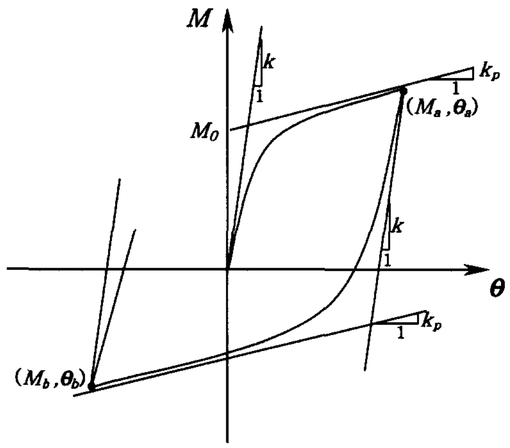

Fig. 4 Loading, Unloading, and Reverse Loading Model of PR Connections

$$f(M,\theta)=0 \tag{4}$$

and its unloading and reloading curve can be described by the following equation:

$$f\left(\frac{M-M_a}{2},\frac{\theta-\theta_a}{2}\right)=0 \tag{5}$$

where (M_a, θ_a) is the load reversal point as shown in Fig. 4.

Using the Masing rule and the Richard model, the unloading and reloading behavior of a PR connection is given by:

$$M=M_a-\frac{(k-k_p)(\theta_a-\theta)}{\left(1+\left|\frac{(k-k_p)(\theta_a-\theta)}{2M_0}\right|^N\right)^{1/N}}-k_p(\theta_a-\theta) \tag{6}$$

and $K_C(\theta)$ for unloading and reloading status is calculated using Equation (6) and can be expressed as:

$$K_C(\theta)=\frac{dM}{d\theta}=\frac{(k-k_p)}{\left(1+\left|\frac{(k-k_p)(\theta_a-\theta)}{2M_0}\right|^N\right)^{\frac{N+1}{N}}}+k_p \tag{7}$$

If (M_b, θ_b) is the next load reversal point, as shown in Fig. 4, the reloading relation between M and θ and corresponding stiffness can be obtained simply by replacing (M_a, θ_a) with (M_b, θ_b) in Equations (6) and (7), respectively. Thus, the proposed method uses Equations (1) and (3) when the connection is loading and Equations (6) and (7) when the connection is unloading and reloading. This represents hysteretic behavior at the PR connections. The basic FEM formulation of the structure remains unchanged and the PR connections can be successfully incorporated for seismic loading with this approach.

2.4 Uncertainties in the Connection Model

For the reliability analysis of structures, all the load and resistance-related parameters need to be explicitly considered since they are expected to have uncertainty in their estimates. In addition, the uncertainties in connection behavior come from uncertainties in the manufacturing and assembly processes and also from modeling uncertainties[15),16)]. Deterministic predictions of connection behavior, based on either empirical formulations or single test data, are likely to overestimate the strength and stiffness of connections. Since, in practice, parameters in a typical M-θ curve are estimated from experimental results using a curve-fitting technique, deterministic curves do not account for the scatter in the connection behavior and a computational model needs to be set up to address the scatter phenomenon adequately. To consider uncertainty in modeling the behavior of PR connections in this study, the four parameters in the Richard model are considered to be the basic random variables as shown in Fig. 5.

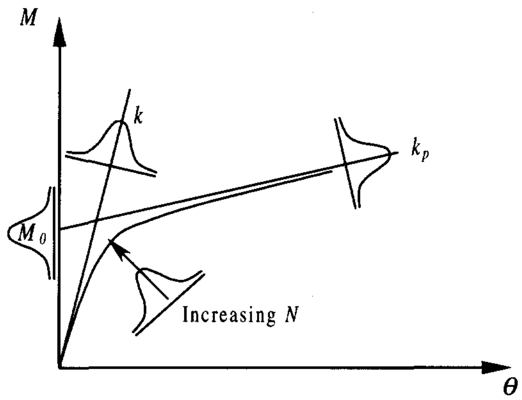

Fig. 5 Random parameters of in the Richard Model

3. Stochastic Finite Element-Based Seismic Risk Estimation

The proposed stochastic finite element-based seismic risk estimation algorithm integrates the concepts of RSM, FEM, FORM, and an iterative linear interpolation scheme. Some of the essential features are discussed below.

3.1 Systematic Response Surface Method

The primary purpose of applying RSM in reliability analysis is to approximate the original complex implicit limit state function[17),18),19)] using a simple explicit polynomial. At least a second order polynomial is necessary for the type of problem under consideration. In this study, two types of second order polynomial(with and without cross terms) are used to represent the response surface. They can be expressed as:

$$\hat{g}(\mathbf{X}) = b_0 + \sum_{i=1}^{k} b_i X_i + \sum_{i=1}^{k} b_{ii} X_i^2 \tag{8}$$

$$\hat{g}(\mathbf{X}) = b_0 + \sum_{i=1}^{k} b_i X_i + \sum_{i=1}^{k} b_{ii} X_i^2 + \sum_{i=1}^{k-1} \sum_{j>i}^{k} b_{ij} X_i X_j \tag{9}$$

where X_i(i=1, 2,..., k) is the i^{th} random variable, and b_0, b_i, b_{ii}, and b_{ij} are unknown coefficients to be determined. The polynomial can be fully defined from regression analysis or by solving a set of simultaneous equations using information on responses obtained at specific data points called sampling points. The selection of sampling points where responses need to be calculated is known as experimental design. Saturated design and central composite design(CCD) could be the two most promising techniques available to generate sampling points. By considering the two design methods and the form of the polynomial, the three response surface models suggested by Huh[1] are considered in this paper. They are Model (1): saturated design using a second order polynomial without cross terms; Model (2): saturated design using a full second order polynomial; and Model (3): CCD using a full second order polynomial. Although the details of these models cannot be presented here due to lack of space, they are available in the literature[1),2),10)].

To accurately and efficiently estimate the probability of failure, it is necessary to improve on the location of the center point in subsequent iterations. Bucher & Bourgund[17] suggested an iterative linear interpolation scheme that can be used to locate the center point efficiently and accurately. It can be mathematically represented as:

$$\mathbf{x}_{C_2} = \mathbf{x}_{C_1} + (\mathbf{x}_{D_1} - \mathbf{x}_{C_1}) \frac{g(\mathbf{x}_{C_1})}{g(\mathbf{x}_{C_1}) - g(\mathbf{x}_{D_1})} \qquad \text{if} \quad g(\mathbf{x}_{D_1}) \geq g(\mathbf{x}_{C_1}) \tag{10}$$

$$\mathbf{x}_{C_2} = \mathbf{x}_{D_1} + (\mathbf{x}_{C_1} - \mathbf{x}_{D_1}) \frac{g(\mathbf{x}_{D_1})}{g(\mathbf{x}_{D_1}) - g(\mathbf{x}_{C_1})} \qquad \text{if} \quad g(\mathbf{x}_{D_1}) < g(\mathbf{x}_{C_1}) \tag{11}$$

where $\mathbf{x}_{C_1}$ and $\mathbf{x}_{D_1}$ are the center point and the checking point for the first iteration, and $g(\mathbf{x}_{C_1})$ and $g(\mathbf{x}_{D_1})$ are the actual responses of the limit state function estimated from dynamic FEM analysis at $\mathbf{x}_{C_1}$ and $\mathbf{x}_{D_1}$, respectively. The point $\mathbf{x}_{C_2}$ can be used as a new center point for the next iteration. This iteration scheme needs to be continued until a preselected convergence criterion is satisfied.

Throughout the iteration process, the three models identified earlier need to be intelligently integrated to achieve computational efficiency and accuracy. The efficiency and accuracy of the algorithm can be increased by applying two promising schemes[1),2)]. They are: Scheme (1): saturated design using a second order polynomial without cross terms for the intermediate iterations and saturated design using a full second order polynomial for the final iteration; and Scheme (2): saturated design using a second order polynomial without cross terms for the intermediate iterations and CCD using a full second order polynomial for the final iteration.

3.2 Limit State Functions for Risk Estimation

Risk has to be estimated with respect to a limit state function. It is broadly divided into two groups, i.e., the serviceability and strength limit states. The proposed algorithm is capable of calculating risk using both types of limit states.

For seismic loading, the serviceability limit state corresponding to overall lateral displacement is considered in this study. The general form of a serviceability limit state can be represented as:

$$g(\mathbf{X}) = \delta_{allow} - y_{max}(\mathbf{X}) = \delta_{allow} - \hat{g}(\mathbf{X}) \tag{12}$$

where δ_{allow} is the allowable overall lateral displacement specified in the applicable design codes and $y_{max}(\mathbf{X})$ is the corresponding maximum overall lateral displacement estimated using the proposed algorithm.

In this study, all the elements in the structural system are considered as beam-columns subjected to axial load and bending moment simultaneously. For the strength limit state, interaction equations suggested by the American Institute of Steel

Construction's(AISC's) Load and Resistance Factor Design(LRFD) manual[4] are thus used in this study. They are:

$$g(\mathbf{X}) = 1.0 - \left(\frac{P_u}{P_n} + \frac{8M_{ux}}{9M_{nx}} \right) = 1.0 - \left[\hat{g}_p(X) + \hat{g}_{M_x}(X) \right]$$

$$\text{if } \frac{P_u}{\phi P_n} \geq 0.2 \qquad (13)$$

$$g(\mathbf{X}) = 1.0 - \left(\frac{P_u}{2P_n} + \frac{M_{ux}}{M_{nx}} \right) = 1.0 - \left[\hat{g}_p(X) + \hat{g}_{M_x}(X) \right]$$

$$\text{if } \frac{P_u}{\phi P_n} < 0.2 \qquad (14)$$

where ϕ is the resistance factor, P_u is the required tensile/compressive strength, P_n is the nominal tensile/compressive strength, M_{ux} is the required flexural strength and M_n is the nominal flexural strength. P_n and M_{nx} can be calculated using AISC's LRFD code. Huh and Haldar[2] discussed the risk evaluation procedure for the two limit states in detail elsewhere.

3.3 Solution Strategy

The solution strategy can be stated as follows. First, the initial center point is assumed to be the mean values of random variables for the first iteration. The responses are calculated by conducting nonlinear FEM at the experimental sampling points for the response surface model, that is, saturated design with the second order polynomial without cross terms. A limit state function is thus generated in terms of k basic random variables. Using the explicit expression for the limit state function and FORM, the reliability index β, and the corresponding coordinates of the checking point and direction cosines for each random variable are obtained. The coordinate of the new center point is obtained by applying the linear interpolation scheme(Equation (10) or (11)). The updating of the location of the center point continues until it converges to a predetermined tolerance level. In the final iteration, the information on the most recent center point is used to formulate the final response surface using either saturated design with a full second order polynomial or CCD with a full second order polynomial depending on the number of random variables. This gives an explicit expression of the limit state function. The FORM method is then used to calculate the reliability index and the corresponding most probable failure point (MPFP). The flowchart that shows the main steps of the proposed algorithm is given in Fig. 6.

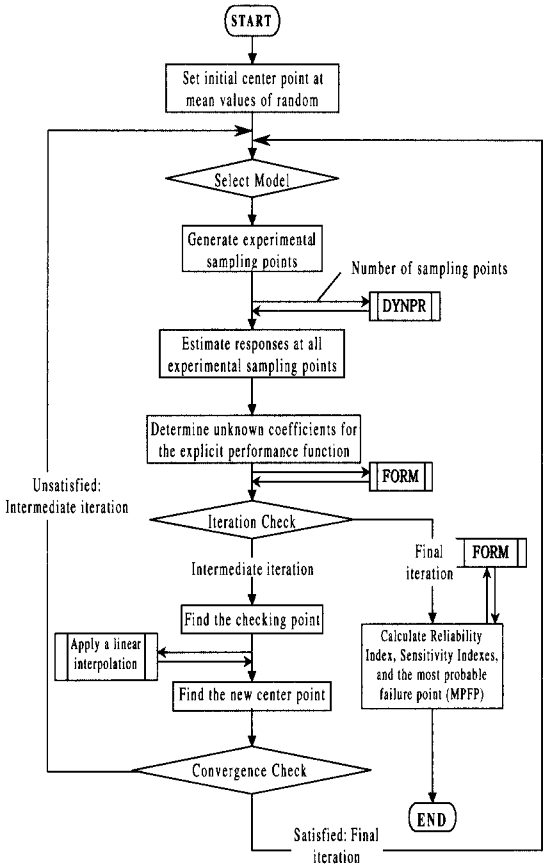

Fig. 6 Flowchart of the Proposed Algorithm

4. Numerical Example

To investigate the effect of flexibility in the connections, the two-story one bay steel frame structure shown in Fig. 7 is considered. The beams and columns of the frame are made of W27×84 and W14×426, respectively, and A36 steel is used. To represent actual seismic loading condition as closely as possible, the frame is excited for 15 seconds by the actual acceleration time history recorded at Canoga Park during the Northridge earthquake of 1994(North-South component) as shown in Fig. 8.

Both the serviceability and strength limit states discussed in Section 3.2 [Equation (12), and Equation (13) or (14), respectively] are considered in this example. For the serviceability limit state, the permissible lateral displacement at the top of the frame is assumed to not exceed $h/400$, where h is the height of the frame. For this example, δ_{allow} becomes 1.905cm. For the strength limit state, the weakest member(beam element at d in Fig. 7) is investigated here.

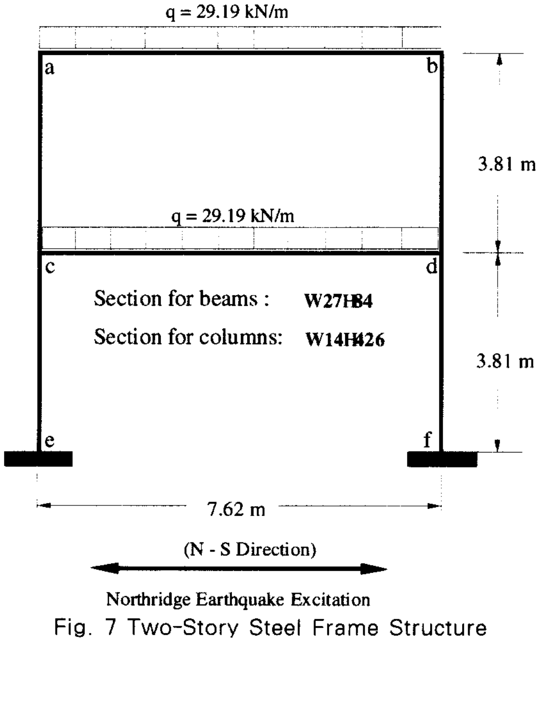

Fig. 7 Two-Story Steel Frame Structure

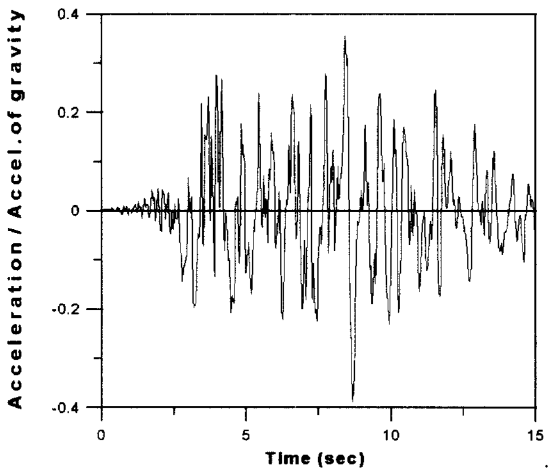

Fig. 8 Northridge Earthquake Time History for 15 seconds(N-S)

The frame is first analyzed assuming the members are connected rigidly(the connections are FR type). This is denoted as Case 1. Then, the four beam-to-column connections at a, b, c, and d shown in Fig. 7 are considered to be partially restrained. In order to consider the effects of different rigidities in the connections, three $M-\theta$ curves(Curve 1, Curve 2, and Curve 3 in Fig. 9) representing very high, intermediate, and very low rigidities, respectively, are considered. They are denoted hereafter as Case 2, Case 3, and Case 4, respectively. The random variables associated with seismic loading and structural resistance including the four parameters of the

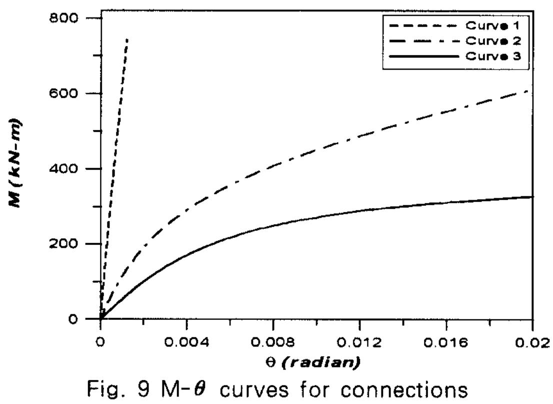

Fig. 9 M-θ curves for connections

Richard model representing the three curves and their probabilistic descriptions, as available in the literature, are listed in Tables 1 and 2 for the serviceability and strength limit states, respectively. The terms ξ and g_e in Tables 1 and 2 stand for the damping coefficient expressed as a percent of the critical damping and a parameter introduced to incorporate the uncertainty in the amplitude of seismic acceleration, respectively. All random variables are assumed to be statistically independent for the numerical calculation.

Using the proposed nonlinear SFEM algorithm, the reliability indexes of the frame are estimated for both limit states of all four cases. Scheme (2) identified in section 3.1 is used to calculate the reliability index for Case 1. Scheme (1) is used to evaluate the reliability index for Case 2, Case 3, and Case 4.

For the verification purpose, the results of Case 1 and Case 2 for both the serviceability and strength limit states are summarized in Table 3 in terms of reliability index, corresponding failure probability, error, CPU time, and sensitivity

Table 1 Statistical Description of the Random Variables for Serviceability

Random Variable	Mean Value				COV	Dist.
	Case 1	Case 2 (Curve 1)	Case 3 (Curve 2)	Case 4 (Curve 3)		
$E(kN/m^2)$	1.9994×10^8				0.06	LN
$A^b(m^2)$	1.600×10^{-2}				0.05	LN
$I^b{}_x(m^4)$	1.186×10^{-3}				0.05	LN
$A^c(m^2)$	8.065×10^{-2}				0.05	LN
$I^c{}_x(m^4)$	2.747×10^{-3}				0.05	LN
ξ	0.05				0.15	LN
g_e	1.00				0.20	Type I
k(kN · m/rad)	N/A	1.13×10^6	1.47×10^5	5.65×10^4	0.15	N
k_p(kN · m/rad)	N/A	1.13×10^5	1.13×10^4	1.13×10^3	0.15	N
M_0(kN · m)	N/A	508.64	452.12	339.09	0.15	N
N	N/A	0.50	1.00	1.50	0.05	N

b=beam, c=column, N/A=Not Available, LN=Log-Normal, N=Normal

Table 2 Statistical Description of the Random Variables for Strength Limit State

Random Variable	Mean Value				COV	Dist.
	Case 1	Case 2 (Curve 1)	Case 3 (Curve 2)	Case 4 (Curve 3)		
$E(kN/m^2)$	1.9994×10^8				0.06	LN
$F_y(kN/m^2)$	2.4822×10^5				0.10	LN
$I^b{}_x(m^4)$	1.186×10^{-3}				0.05	LN
$Z^b{}_x(m^3)$	3.998×10^{-3}				0.05	LN
$I^c{}_x(m^4)$	2.747×10^{-3}				0.05	LN
ξ	0.05				0.15	LN
g_e	1.00				0.20	Type I
k(kN · m/rad)	N/A	1.13×10^6	1.47×10^5	5.65×10^4	0.15	N
k_p(kN · m/rad)	N/A	1.13×10^5	1.13×10^4	1.13×10^3	0.15	N
M_0(kN · m)	N/A	508.64	452.12	339.09	0.15	N
N	N/A	0.50	1.00	1.50	0.05	N

b=beam, c=column, N/A=Not Available, LN=Log-Normal, N=Normal indexes of the random variables. The results are compared with Monte Carlo simulation(MCS) using 100,000 simulations, also shown in Table 3. A super computer(SGI Origin 2000) was used for the numerical calculation for both the proposed algorithm and MCS. Table 3 shows that the probability of failure for both limit states of the two cases is very close to the MCS results and the ratio of CPU time for MCS to that of proposed algorithm ranges from 476 to 837. The proposed algorithm is therefore viable and efficient for the reliability analysis of nonlinear structures with PR connections subjected to seismic loading.

The behavior of the frame with rigid and PR connections can be investigated by simply comparing the results given in Table 4, which show the reliability indexes for both limit states of all four cases. The reliability indexes for the serviceability limit state decreased significantly

Table 3 Reliability Analysis Results

Limit State		Serviceability		Strength Limit State	
Case		Case 1	Case 2	Case 1	Case 2
Monte Carlo Simulation	P_f	0.02887	0.10244	0.00002	0.00896
	CPU Time(sec)	98183	107571	72949	106003
Proposed Algorithm	Scheme	(2)	(1)	(2)	(1)
	β	1.914	1.274	4.098	2.351
	P_f	0.027792	0.10133	0.000021	0.00936
	CPU Time(sec)	182.8	131.0	153.3	126.7
	Error*	3.73 %	1.08 %	-5.00 %	-4.46 %
Sensitivity Index γ	E	-0.2389	-0.3515	-0.0826	-0.0745
	A^b	0.0109	0.0003	-	-
	I^b_x	-0.1290	-0.0785	0.0062	0.0053
	Z^b_x	-	-	-0.1785	-0.1667
	A^c	0.0349	0.0142	-	-
	I^c_x	-0.1118	-0.1272	-0.1183	-0.1243
	Z^c_x	-	-	-	-
	F_y	-	-	-0.3607	-0.3185
	ξ	-0.2792	-0.2670	-0.2589	-0.2438
	g_e	0.9135	0.8825	0.8661	0.8279
	k	N/A	-0.0367	N/A	-0.3182
	k_p	N/A	-0.0478	N/A	-0.0013
	M_0	N/A	-0.0084	N/A	-0.0484
	N	N/A	-0.0066	N/A	-0.0384

b=beam, c=column, N/A=Not Available, Error* based on P_f

Table 4 Reliability Analysis Results

Limit State	FR Connection	PR Connections		
	Case 1	Case 2(Curve 1)	Case 3(Curve 2)	Case 4(Curve 3)
Serviceability Limit State	$\beta=1.914$	$\beta_1=1.274$	$\beta_2=-0.008$	$\beta_3=-0.899$
Strength Limit State (Beam Element at d)	$\beta=4.098$	$\beta_1=2.351$	$\beta_2=2.558$	$\beta_3=3.156$

with the decrease in the rigidity of the PR connections. The frame became very weak in serviceability, particularly when PR connections were represented by very flexible Curve 3. This is expected since the PR connections make the frame more prone to failure than FR connections as in Case 1. Due to the redistribution of moment in beam c-d, the reliability indexes for the strength limit state also changed. However, the frame is much more prone to failure in serviceability than in strength, that is, the reliability indexes for the serviceability limit state are much smaller than those for the strength limit state, in most cases. This implies that an unbraced steel frame is more vulnerable to failure caused by lateral displacement than by strength. The allowable lateral deflection considered in the study, i.e., $\delta_{allow} = (h/400)$, is appropriate for the static loading. For dynamic and seismic loading this may be a very conservative value.

From the example considered in this paper, it is clear that the consideration of appropriate connection rigidities and the uncertainty associated with modeling them significantly affects the reliability estimation of steel frame structures. Therefore, it can be concluded that connection rigidity should be taken into account appropriately in reliability analysis of steel frame structures subjected to seismic loading.

5. Conclusions

An efficient and accurate algorithm is proposed to estimate the reliability of structures with PR connections subjected to seismic loading in the time domain. The flexibility of connections is represented by the four-parameter Richard model. The uncertainties in the loading and resistance-related parameters and the parameters in the Richard model are incorporated in the algorithm. The earthquake loading can be applied in the form of time histories to excite structures, enabling realistic consideration of earthquake loading. The proposed algorithm intelligently integrates the concepts of the finite element method, the response surface method, the first order reliability method, and the iterative linear interpolation scheme. The iterative scheme suggested here improves computational efficiency considerably.

With the help of an example, some of the following conclusions can be made. The proposed algorithm can be used to evaluate the risk associated with steel frame structures with and without PR connections subjected to seismic loading for both the serviceability and strength limit states. It appears to be a very effective, accurate and robust algorithm. The presence of PR connections reduces the overall stiffness of a steel frame structure and adds a major source of energy dissipation in it, leading changes of its dynamic characteristics. As a result, considering PR connections and the uncertainties associated with modeling them significantly affects reliability estimation for steel frame structures. Also, the serviceability limit state appears to be the critical limit state for seismic loading. Consequently, it may be stated that the common practice in the profession of designing the frame for the strength limit state with consideration of all connections to be rigid is not appropriate and may need to develop stricter and more realistic code requirements for a steel frame structure, particularly for aseismic design.

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