A SIMPLE PROPOSAL FOR ULTIMATE SEISMIC DEMAND
EVALUATION OF MOMENT RESISTING STEEL FRAMES

鋼構造ラーメン骨組の終局地震荷重効果評価手法の簡略化

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ABSTRACT: A simple proposal for ultimate seismic demand evaluation of multi-degree-of-freedom moment resisting steel frames is developed from limit analysis and first order reliability method (FORM). First, a procedure is proposed for 'vibration mode-failure mode' integrated analysis, wherein the restoring force characteristics are represented by a global convex yield polyhedron model, instead of a set of member hysteresis based models usually adopted in the inelastic structural analysis. Then, FORM is extended to seismic design in the form of 'random pushover analysis', to choose an appropriate number of failure modes to be considered in the vibration mode-failure mode analysis. Random pushover shows a positive correlation with the deterministic analytical procedures and can be further used to determine the lateral load patterns for pushover by the nonlinear static procedures. A six-storey two-bay frame is taken for case study and is analysed by the proposed methods for three ground motions and an impulse. By comparing the results with the member-hysteresis based analytical procedures, this study indicates that the proposed simple methods can serve as a good tool for seismic demand evaluation for practical purposes.

Keywords: demand evaluation, inelastic response, yield polyhedron, planar steel moment resisting frame, design point

要求評価、弾塑性応答、降伏多面体、鋼構造平面ラーメン、設計点

INTRODUCTION

The emphasis in seismic resistance design is shifting from 'strength' to 'performance' following the damage and collapse of numerous structures during recent earthquakes. Gradually, performance based designs are becoming a part of code provisions with the publication of FEMA-273 [1] in USA and Enforcement Order and Regulations after Building Standard Law of Japan was revised in 1998 [2]. At the same time, it may be fair to say that simple and efficient methods for capturing the essential/important features affecting the performance have not been adequately developed. The research summarised in this paper focuses on the development of design friendly methods for seismic demand evaluation of multi-degree-of-freedom moment resisting steel frames. As a simplification over member hysteresis based analytical procedures, an approach for vibration mode-failure mode integrated analysis is proposed, wherein the failure modes are obtained by the limit analysis of the structure. Random pushover approach, based on FORM, is presented to choose an appropriate number of important/dominant failure modes, for structures having large number of failure modes, for further consideration in the vibration-mode failure mode analysis. A suggestion is made on the issue of

load pattern selection, for detection of weak/ local collapse mechanism by nonlinear static pushover procedures, through examination of the design points of the failure modes. Application of the proposed methods is discussed for a six-storey two-bay frame by studying its response to El Centro (NS), Hachinohe (EW), Fukiai (NS) ground motions and an impulse. Considering the practical difficulties in explaining the physical meaning of some of the proposed ideas, a two-storey single-bay frame is also taken to illustrate the basic principles. Results are compared with the member-hysteresis based detailed analytical models to judge the validity of the proposed simplified methods.

DESCRIPTION OF FRAME MODELS

A six-storey two-bay frame [3] is taken to study the application of the proposed concepts. The storey height is 3.75m and span of each bay is 6m. The frame is shown in Fig. 1 and the member properties and floor weights are shown in Table 1 and 2, respectively. The modulus of elasticity and yield stress for steel is taken as 205 GPa and 235 MPa, respectively. A constant modal-damping ratio of 2% is considered for all the vibration modes. The vibration periods are shown in Table 3.

Note: A part of this research was presented in Ref. [13].

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The failure modes considered under horizontal loads are shown in Fig. 2 and have been drawn based on the experience. These include local storey collapse mechanisms and combined mechanisms. Locations of the plastic hinges are assumed at the top and bottom ends of the columns, which have undergone side sway, and at the floor beams end connected with the side columns. At the junction of the floor beams and the central column, plastic hinges are assumed in the beams or in the columns, whichever is having the least summation of the plastic moment capacities at the junction.

The total weight of the frame is 3326.4 kN and yield base shear coefficient is 0.11. The frame is analysed for El Centro (NS), 1940, Hachinohe (EW), 1968, Fukiai (NS), 1995 ground motions and an impulse

of 0.75 m/sec velocity. The details of the ground motions are shown in Table 4.

Basic principles are illustrated with the help of a two-storey single-bay frame as shown in Fig. 3. The ratio of span (L) to storey height (H) and mass at 2nd floor to roof are 1.6 and 1.5464, respectively. The relative member properties are shown in Table 5. The total weight (W) of the frame is 484 kN. 2% modal damping is assumed for both vibration modes. The elastic vibration periods for the first and second mode are 1.23 and 0.472 seconds, respectively. The frame is analysed for El Centro (NS), 1940 ground motion with peak ground acceleration as 0.3m/sec². The failure modes are shown in Fig. 4.

Fig.1 Six-Storey Two-Bay Frame

Fig. 2 Collapse Modes for six-storey two-bay frame

Table 1 – Member Properties

Mem. ID	Moment of Inertia (10 ⁻⁸ m ⁴)	Plastic Moment Capacity (kN. m)
1	2492	83.19
2	8091	194.35
3	5696	151.11
4	11260	247.46
5	14920	301.51
6	23130	307.15
7	16270	239.47
8	11770	188.94
9	8356	147.58
10	3892	86.25

Fig. 3 Two Storey Single Bay Frame

Fig. 4 - Failure Modes of the two-storey frame

Table 2–Floor Weights

Floor	Weight (kN)
Roof	380.4
6	589.2
5	589.2
4	589.2
3	589.2
2	589.2

Table 3- Vibration Periods (Sec)

Mode	Time Period
1	2.57
2	0.98
3	0.57
4	0.43
5	0.31
6	0.23

Table 4 – Details of Ground Motions

Ground Motion	Earthquake	Peak Ground Acceleration (in gravity acceleration)	Duration (Sec)
El Centro (NS)	Imperial Valley, 1940	0.325	10
Hachinohe (EW)	Tokachi - Oki, 1968	0.186	30
Fukiai (NS)	Hyogoken – Nanbu, 1995	0.700	20

Table 5- Member Properties

Property ratio	Value
Mpb1/Mpc	1.179
Mpb2/Mpc	1.036
Ib1/Ic	3.353
Ib2/Ic	1.561

Mp– Plastic moment capacity
I–Moment of Inertia

VIBRATION MODE-FAILURE MODE ANALYSIS

In order to visualise failure modes of a structure in a nonlinear seismic analysis, a simple procedure for vibration mode-failure mode integrated analysis is proposed. The response of the structure is confined within a convex region bounded on all sides by the failure planes (referred to as yield polyhedron hereafter). Fig. 5 shows yield polyhedron for the two-storey frame. The f_1 and f_2 represent the restoring forces (in original space) at the second floor and roof level, respectively. Representation of the restoring force characteristics of the structure by a yield polyhedron model considerably reduces the magnitude of the analysis, compared with member-hysteresis based models usually employed in the inelastic analysis. The likely failure modes can be drawn based on the experience or stochastic limit analysis using compact procedure [4, 5] may be used for complex situations. Sequence of yielding before mechanism formation i.e. partial yielding is ignored in the yield polyhedron model. As the analysis is performed in the modal coordinates, the failure modes can be transformed to the modal coordinates using the following transformation equation:

$$\{f\} = [\phi]^T \{r\} \quad (1)$$

where, $\{r\}$, $\{f\}$ and $[\phi]$ are modal restoring force vector, original restoring force vector and modal participation matrix, respectively. Participation matrix $[\phi]$ is obtained from the modal analysis of linear elastic system and is normalised such that the participation factor for any mode is unity. Each column vector of $[\phi]^T$ represents the lateral loading pattern of each vibration mode and remains constant during the analysis. Equation of motion for seismic excitation is integrated by numerical methods, such as the central difference method, in the modal coordinates. The locus of restoring forces can move within or tangentially on the boundary of the yield polyhedron, as shown in Fig. 6 for the two-storey frame. The r_1 and r_2 are components of $\{r\}$ and represent the restoring forces for the first and second mode, respectively.

In the inelastic state, the plastic component of the deformation is computed from the normality rule of plasticity.

Yield polyhedron approach enables a quick examination of the seismic response in terms of the failure modes of the structure, which is quite informative and helpful in understanding critical/ important design situations and in taking remedial measures.

Fig. 5 Global Yield Polyhedron in Original Restoring Force Space

RANDOM PUSHOVER ANALYSIS

Seismic Loading Model

As the ground motion varies randomly in time, mean value or expected value of modal restoring forces is taken as zero and the vibration modes are considered independent. In usual notations:

$$E(r_j) = \bar{r}_j = 0; \quad E(r_i r_j) = 0 \quad (i \neq j) \quad (2)$$

$$\sigma_{r_j} = c \cdot m_j^* S_{a_j} \quad (3)$$

where, r_j , m_j^* and σ_{r_j} are the modal restoring force, effective modal mass and standard deviation of the modal restoring force, for the j th vibration mode, respectively. c is a constant which is determined from the relationship between the standard deviation of modal restoring force at arbitrary points in time and its mean extrema and is taken as one-third for all the modes [6, 7]. S_{a_j} is the ordinate of acceleration response spectra, which is dependent on period and damping of the j th vibration mode (refer Fig. 7).

Reduction of Failure Modes

Stochastic limit analysis, based on FORM, is applied for identification of more likely failure modes. It is assumed that the loads and resistances are independent of each other. The performance function (g) for a failure mode can be written in terms of the work done by the resistance capacity and load effects as:

$$g = \sum_{i=1}^m M_{pi} |\theta_{pi}| - \sum_{j=1}^n a_j r_j = g_0 - \sum_{j=1}^n a_j r_j \quad (4)$$

where, m and n are number of element resistance (moment) capacity and number of vibration modes respectively. The work done by the resistance capacity is represented by $\sum_{i=1}^m M_{pi} |\theta_{pi}|$, where M_{pi} is the basic variable of element moment capacity and $|\theta_{pi}|$ represents the rotation of the corresponding plastic hinge. The work done by the load effects is expressed in the modal coordinates and is represented by $\sum_{j=1}^n a_j r_j$, where a_j is the j th modal component of plastic deformation compatible with before mentioned plastic hinge rotations.

Fig. 6 Locus of Restoring Force and Design Points in Modal Space

$$\text{Mean value of } g \quad \mu_g = \sum_{i=1}^m \bar{M}_{pi} |\theta_{pi}| - \sum_{j=1}^n \bar{r}_j a_j \quad (5)$$

$$\text{Standard deviation} \quad \sigma_g = \sqrt{\sum_{i=1}^m \sigma_{mpi}^2 \theta_{pi}^2 + \sum_{j=1}^n (\sigma_{rj})^2 (a_j)^2} \quad (6)$$

$\bar{M}_{pi}$, $\bar{r}_j$ are mean values and σ_{mpi} , σ_{rj} are standard deviations of load and resistance capacity, respectively.

$$\text{Reliability index} \quad \beta = \frac{\mu_g}{\sigma_g} = \frac{g_0}{\sigma_g} \quad (7)$$

A lower reliability index indicates higher probability of failure. In our view, failure modes with probability less than about 10% of the most likely ($\beta_{\min}$) failure mode can be excluded from further analysis. Alternatively, failure modes with $\beta \geq \beta_{\min} + \Delta$ can be neglected. For Gaussian distribution if $\beta_{\min} \geq 1.4$, Δ equal to unity would be adequate. In addition, the maximum number of failure modes considered should preferably be limited to the number of vibration degrees of freedom, to avoid possibility of ill conditioning during dynamic analysis. For structures with very large numbers of potential failure modes, mutual correlation between the failure modes should be considered, as many of them may be partially correlated. PNET [8], which stands for probabilistic network evaluation technique was applied for the six-storey frame. The correlation coefficient between the two failure modes a and b , whose performance functions are denoted by $g_a = g_{0a} - \sum_{j=1}^n a_j r_j$, $g_b = g_{0b} - \sum_{j=1}^n b_j r_j$ respectively, can be written as:

$$\rho_{ab} = \frac{E[(g_a - E(g_a))(g_b - E(g_b))]}{\sigma_{ag} \sigma_{bg}} \quad (8)$$

where, σ_{ag} , σ_{bg} are standard deviations of g_a , g_b respectively. Resistance capacity has been considered deterministic in the analysis. Under such a condition, equation (8) can be further simplified as:

$$\rho_{ab} = \frac{\sum_{j=1}^n a_j b_j \sigma_{rj}^2}{\sqrt{\left(\sum_{j=1}^n a_j^2 \sigma_{rj}^2 \right) \left(\sum_{j=1}^n b_j^2 \sigma_{rj}^2 \right)}} \quad (9)$$

Failure modes with correlation coefficients more than a demarcating correlation ρ_0 are unified and represented by a single failure mode having lowest reliability index among them. As the failure probabilities range from 10^{-1} to 10^{-3} , $\rho_0 = 0.70$ appear appropriate [8]. However, adoption of such a low value resulted in the elimination of the majority of the failure modes for the six-storey frame. A value of $\rho_0 = 0.97$ was found to give satisfactory result. Table 6 and 7 lists the reliability index of the failure modes for the six-storey and two-storey frame, respectively. Table 8 lists the failure modes obtained by PNET for the six-storey frame.

Lateral Load Pattern by Design Point of Failure Modes

Issue of load pattern selection is critical for performance evaluation by nonlinear static pushover procedures. At present, pushover by using invariant load patterns such as proportional to first mode shape or a uniform load pattern is in practice [9]. Static pushover analysis using such load patterns may lead to misleading predictions particularly when localised yielding mechanism have formed. Some adaptive load patterns [10] have been suggested, but none have proven to be universally applicable, as

higher mode effects are not accounted. On the other hand, load pattern obtained from the design point of failure mode, as explained next, appear to be free from such limitations.

Design point of a failure mode is the most likely failure point on its surface. Fig. 8 shows the design points of the failure modes in relation to reliability index for the two-storey frame. The design point of the most likely failure mode represents the design point of the yield polyhedron and this design point lies on the surface of the yield polyhedron e.g. design point of first storey failure mode in Fig. 8. The design points for some failure modes may not lie on the surface of the yield polyhedron such as the design point for the second storey collapse as shown in Fig. 8. Most likely failure point of a failure mode lies with minimum distance from origin [11] in the standard variable space. Standard variables are random variables with mean value as zero and standard deviation being unity and can be written as:

$$u_{mpi} = \frac{M_{pi} - \bar{M}_{pi}}{\sigma_{mpi}}, \quad u_{rj} = \frac{r_j - \bar{r}_j}{\sigma_{rj}} \quad (10)$$

Performance function in terms of standard variables can be written as:

$$G = \sum_{i=1}^m \left(\sigma_{mpi} u_{mpi} + \bar{M}_{pi} \right) |\theta_{pi}| - \sum_{j=1}^n \left(\sigma_{rj} u_{rj} + \bar{r}_j \right) a_j \quad (11)$$

Design point u^* is the closest point from origin in the standard variable space, i.e.

$$u_{mpi}^* = -\beta \frac{\sigma_{mpi} |\theta_{pi}|}{\sigma_g}, \quad u_{rj}^* = \beta \frac{\sigma_{rj} a_j}{\sigma_g} \quad (12)$$

Components of design points in the original space are

$$M_{pi}^* = \sigma_{mpi} u_{mpi}^* + \bar{M}_{pi}, \quad r_j^* = \sigma_{rj} u_{rj}^* + \bar{r}_j = \sigma_{rj} u_{rj}^* + \frac{\beta}{\sigma_g} \left(\sigma_{rj} \right)^2 a_j \quad (13)$$

Fig. 6 shows the design points of the failure modes for the two-storey frame. It can be seen that the design point for the most likely failure mode, i.e. first storey, is traced by the locus of modal restoring force in the dynamic analysis. As it is difficult to visualise the similar situation for the six-storey frame, the coordinates of five nearest points (on the loci of the modal restoring force) from the design point of fifth storey (most likely) collapse are plotted. Fig. 9 shows such a plot for the El Centro ground motion. For the purpose of clarity, only first four modal restoring force coordinates (r_1 through r_4) are plotted for the design point and five nearest points in Fig. 9. Table 9 shows the coordinates of the fifth storey design point, nearest and average of five nearest (from the design point) on the loci, in terms of the modal restoring forces. By examining Fig. 9 and Table 9, it is fair to say that the loci in the dynamic analysis passed close to the design point. The coordinates of the design point shear, as listed in Table 9, indicate dominance of the first two modes. Similar results were obtained for other ground motions. These observations suggest that design point represents a realistic load profile and also incorporates higher mode effects.

The load profiles in the original coordinates can be obtained by using equation (1) and the coordinates of the design point in the modal restoring force space. Design point of the fifth storey collapse was used as the initial load profile in the compact procedure. At the end of the iterations, same collapse mechanism was observed which is shown in Fig. 10. This suggests that pushover using design point loading of the most likely collapse mechanism would be able to detect it and therefore, design point loading profile can be used for the purpose of nonlinear static pushover analysis.

Fig. 7 Random Loading Model

Fig.8 Design Point of Failure Modes

Fig. 9 Load Profile Near Design Point of Fifth Storey Collapse (El Centro)

Fig. 10 Failure Mode Traced By Compact Procedure

Table 6 Reliability Index for Failure Modes of Six-Storey Frame

Failure Mode	Reliability Index β			
	El Centro	Hachinohe	Fukiai	Impulse
1	1.912	1.232	1.439	2.026
2	1.943	1.204	1.450	2.080
3	1.939	1.152	1.437	2.034
4	1.986	1.142	1.475	2.037
5	1.955	1.102	1.455	1.977
6	2.110	1.184	1.574	2.127
7	2.189	1.324	1.618	2.261
8	2.020	1.178	1.498	2.063
9	2.011	1.147	1.505	2.036
10	1.865	1.051	1.395	1.878
11	2.054	1.156	1.530	2.070
12	2.392	1.398	1.799	2.367
13	2.107	1.228	1.609	2.129
14	1.803	1.050	1.358	1.851
15	2.027	1.186	1.505	2.098
16	2.715	1.668	2.088	2.723
17	1.861	1.154	1.387	1.982
18	2.097	1.324	1.539	2.288
19	1.560	1.057	1.138	1.712
20	1.911	1.360	1.416	2.230
21	2.273	1.982	1.835	2.623

Table 7 Reliability Index for Failure Modes of Two-Storey Frame

Failure Mode	Reliability Index
First Storey	1.67
Second Storey	2.68
Combined mode	1.79

Table 8 Failure Modes by PNET For Six-Storey Frame

Seismic Motion	Failure Modes
El Centro	19,14,17,10,20,1
Hachinohe	14,19,5,3,2,21
Fukiai	19,14,17,10,3,1
Impulse	19,14,5,17,1,8

Table 9- Design Point Coordinates for Fifth Storey Local Collapse and Dynamic Analysis Loci in the Vicinity (El Centro)

Modal restoring force	Design Point Coordinates (kN)	Nearest Point on Loci (kN)	Average of five nearest on Loci (kN)	Design Point Shear (kN)
R1	-170	-152	-152	85
R2	105	103	103	79
R3	0	11	11	0
R4	-5	-18	-18	4
R5	2	3	3	1
R6	0	2	2	0
Distance from origin (kN)				
	200	184	184	

COMPARISON OF RESULTS

The six-storey frame is analysed for El Centro (NS), Hachinohe (EW), Fukiai (NS) ground motions and an impulse of 0.75 m/sec velocity, by the proposed methods, referred to as RA hereafter. The impulse level is selected from consideration of similar order of deformation pattern in comparison with other ground motions. The energy input spectra and acceleration response spectra are shown in Figs. 11 and 12, respectively. To confirm the validity of RA, results have been compared with a member hysteresis based analysis, performed using a computer program DIANA [12] with all members being idealised as elastic perfectly plastic. As there is vast amount of data, response histories of the top storey, the bottom storey and the most damaged storey i.e. fifth have shown for comparison. Fig. 13 shows the response histories of the storey drifts. Peak response values and time variations are well simulated.

Fig. 11 Energy Input Spectra (in velocity expression)

Fig. 14 compares the hysteresis loops of the storey shear v/s storey drifts. RA provides close approximations to DIANA. However, little shifts in the storey drifts appear in some cases. Fig. 15 compares the envelopes of floor displacements, storey drifts and storey shears. Peak floor displacements are well simulated. Fifth storey appear as the weak/ local collapse mechanism, which coincides with the most likely failure mode indicated by the random pushover analysis. RA has provided consistently good results for the peak fifth storey drift, which is the most important response quantity. For other storey drifts, simulation by RA can be considered satisfactory. In general, peak storey shear estimation appears to be conservative. As an overall view, the proposed methods provide consistently good prediction of response quantities in all cases of seismic excitations.

Fig. 12 Acceleration Response Spectra

Fig. 13 Response History of Storey Drifts

Fig. 14 Hysteresis Loops of Storey Shear v/s Storey Drift

Fig. 15 Envelope of Response Quantities

CONCLUSIONS

A simple and design friendly, seismic demand evaluation procedure is proposed for multi-degree-of freedom moment resisting steel frames. The salient features are as follows:

1. A simplified global yield polyhedron model represents the restoring force characteristics of the structure.
2. Random pushover approach is proposed to choose stochastically dominant failure modes.
3. The lateral load profile obtained from the design point of the most likely failure mode can be used for pushover in the nonlinear static procedures.

The validity of the proposed concepts was judged by comparing the response of a six-storey two-bay frame, to El Centro (NS), Hachinohe (EW), Fukiai (NS) ground motions and an impulse, with member hysteresis based analysis procedures. The proposed procedure provided consistently good results for the response quantities of interest.

The ideas proposed in this paper lay emphasise on simplicity as well as generality and are expected to help the practising engineer, who has limited resources and analytical tools to carry out his responsibilities.

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1. はじめに

構造設計の性能規定化が提唱されている。米国ではすでに FEMA-273 に性能設計の概念を取り入れており、日本でも 1998 年の建築基準法の改定に伴い、様々な性能検証ルートが許容されるようになったが、構造物の各種の能力 (Capacity) が要求性能 (Demand) を満足しているかどうかを検証するという枠組みは、様々な検証法で共通と考えられる。本研究は、鋼構造ラーメン骨組に対する要求性能、特に終局地震荷重効果を動的解析によって評価する手法を、グローバルな視点で大幅に簡素化する解析手法を提案するものである。

本研究で提案する手法は、鋼構造多層ラーメン骨組の安全領域を凸降伏多面体モデルで表し、振動方程式を古典的規準振動モード座標で解く簡略化応答解析手法である。多層骨組の膨大な崩壊モードについては、考慮すべき重要な崩壊モードを、ランダム複合モード載荷下での近似信頼性解析法 (FORM) に基づく確率極限解析を予め実行しておくことにより、崩壊モード数を適切な数に低減しておく手法を提案する。この予備解析手法は、多層骨組の静的弾塑性増分解析において、高次振動を適切に考慮した荷重パターンを設定するためにも用いることができる。

2. 解析対象の鋼構造ラーメン骨組モデル

研究解析対象骨組は文献 3) の Benchmark study に用いられている、層高 3.75m、スパン 6m の 6 層 2 スパン鉄骨造骨組である。弾性 1 次固有周期は 2.57 秒、降伏ベースシア係数は 0.11 及び減衰は全振動モードにおける 2% のモード減衰を仮定した。対象骨組の諸元を図 1 及び表 1、2、3 に示す。

部材耐力を確定としているので、水平荷重に対して考慮する 6 層骨組の崩壊メカニズム形を図 2 に示すものに限定する。また提案する解析手法の物理的意味を簡明に説明するため、2 層 1 スパン骨組モデルの例題を示し、その諸元及び崩壊モードを図 3、4、表 5 に示す。

3. 振動モードと崩壊モードを同時に考慮した応答解析

従来の部材レベルの骨組地震応答解析では、各部材毎に復元力特性モデルを仮定して、応答解析を遂行するので計算が複雑になる。しかし本研究で提案する手法は、骨組の安全領域を、崩壊メカニズム形を重要なもののみに限定した凸降伏多面体モデルでモデル化し、それを、式 (1) より通常座標からモード座標に変換して、応答解析を行う。

図 5 に前述の 2 層 1 スパン骨組の凸降伏多面体モデルを通常の各階床位置の復元力 (水平力) 座標で示している。この凸降伏多面体モデルを用いて行った El Centro (NS) 1940 に対する応答解析結果を、図 6 にモード復元力空間での復元力点の軌道で表している。本解析では、一般化された完全弾塑性モデルが使用され、上記の降伏多面体の内部では弾性挙動が仮定されるとともに、降伏面上の塑性変形増分は降伏面の法線則にしたがう。

4. ランダム・ブッシュオーバー解析 ランダム複合モード荷重モデル

弾性地震荷重効果の静的荷重モデルとして、古典的規準モードの荷重基底ベクトルにモード乗数を乗じて線形結合したランダム複合モード荷重モデルを仮定する。モード乗数の平均値は 0 とし、標準偏差は式 (3)、各次の有効質量と設計用加速度応答スペクトル値の積に比例させる。比例定数は、確率過程の任意時点における標準偏差とある継続時間内の平均最

大値との関係を参考にして、暫定的に $1/3$ と仮定している。また異なるモード乗数間の相関は考慮していない。

崩壊モードの省略

ここでは、崩壊メカニズム形成を仮の限界状態として、各崩壊メカニズムに対する性能関数として式 (4) を用いて解析を行う。また、性能関数の平均値と標準偏差は、荷重と部材耐力の平均値と標準偏差を用いて式 (5)、(6) により評価し、各崩壊メカニズムに対する性能関数の信頼性指標 β を式 (7) によって求める。なお本論文の範囲では、部材耐力を確定とする。前述の 6 層 2 スパン骨組に対して、表 4 に示す 4 種の地震波とインパルス波に対応する 5 種類の加速度応答スペクトルによって各次の荷重モード乗数の標準偏差を設定して行った予備解析結果を表 6 に示している。

本予備解析では、崩壊メカニズムに対応する信頼性指標値が、最も小さい指標値に 1 を加えたもの以上となるような崩壊メカニズムを暫定的に省略している。この設定は望ましい簡略化の程度に依存する。

また骨組の規模が大きく崩壊メカニズムの数が多い場合では、似通った崩壊メカニズムの性能関数は大きな相関を持つ。本解析では PNET 法を用いて、極めて相関の高い崩壊メカニズムは統合化して解析を行う。式 (9) によって 2 つの崩壊メカニズムの相関係数を求め、相関係数の値が 0.97 以上となるものを暫定的に統合化している。表 8 に、以上の条件を満足する最終的に考慮すべき崩壊メカニズムを地震波別に示している。いずれの場合も図 2 に示した 21 個の崩壊メカニズムを 6 つの崩壊メカニズムに集約することになる。

設計点による荷重パターンの選定 (静的弾塑性解析用)

上記の予備解析は、静的弾塑性増分解析における荷重パターンの選定にも役立てることができる。すなわち、骨組に発生しやすい崩壊メカニズムに応じて、その崩壊メカニズムが発生するとすれば最も局所的な確率密度の高くなる荷重パターンを FORM における設計点として評価する。荷重モード乗数の標準空間において設計点を求めるプロセスを式 (10) から式 (13) に示しているが、2 層 1 スパン骨組の場合の各メカニズムに対応する設計点を図 8 に示している。6 層 2 スパン骨組については、El Centro 地震波に対して信頼性指標値が最小となる No.19 の崩壊メカニズムの設計荷重パターンが、応答解析においても実現していることを図 9、表 9 で確認している。

解析結果の比較 (動的弾塑性応答解析)

上記のように崩壊モード数を 6 個に低減した凸降伏多面体モデルによる簡略化応答解析結果と、部材レベルで通常の塑性ヒンジの発生を考慮した骨組地震応答解析による応答解析結果との比較を行う。4 つの入力地震波及びインパルス波の応答スペクトルを図 11、12 に示す。図 13 に層間変位の時刻歴、図 14 に各層の履歴曲線、図 15 に最大相対変位、最大層間変位及び最大層せん断力の分布を比較しているが、若干の誤差は認められるものの、大局的にはよく一致している。

5. まとめ

本研究では鋼構造ラーメン骨組における終局地震荷重効果をグローバルな視点で簡略に評価する応答解析手法を提案した。提案した解析手法を 6 層 2 スパン骨組に適用し、その結果を従来の部材モデルによる詳細な弾塑性地震応答解析結果と比較し、そのグローバルな弾塑性応答を追跡できることを確認した。

(2000年11月10日原稿受理, 2001年2月14日採用決定)