PLASTIC SKELETON SHIFTING CRITERIA FOR ACCEPTANCE EVALUATION OF STEEL MOMENT RESISTING FRAMES

塑性骨格曲線移動則による鋼構造ラーメン骨組の性能判定

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ABSTRACT: This paper focuses on damage simulation for a simplified acceptance evaluation of multi-degree-of-freedom moment resisting steel frames. A global reduced analysis procedure, previously proposed by the authors, is utilised for seismic demand prediction at local (member) level, which are compared with a detailed member based analysis. A plastic skeleton shift damage model is used to predict member cyclic plastic deformation capacity, from the monotonic capacity. The shift parameter is calibrated using the past experimental results on constant amplitudes cyclic loading for H-shaped steel sections. For simplicity of acceptance check, the deformation level corresponding to member strength deterioration, below plastic limit, is considered as critical. A six-storey two-bay frame is studied for three ground motions and an acceleration impulse. This study indicates that the proposed simple methods can serve as a good tool for seismic acceptance evaluation for practical purposes.

Keywords: acceptance evaluation, cumulative damage, inelastic response, planar steel moment resisting frame

性能評価、累積損傷、弾塑性応答、鋼構造平面ラーメン

1. INTRODUCTION

The need for seismic performance evaluation of existing and new constructions has come into focus following damage and collapse of numerous structures during recent earthquakes. Recognising the same, the building design codes in USA [1] and Japan [2] have been revised to introduce explicit concepts of performance based designs. At present, it may be fair to say that simple and efficient methods for performance evaluation have not been adequately developed. With these considerations, a design friendly reduced analysis (RA) approach for ultimate (global) demand evaluation of multi-degree-of-freedom steel moment resisting frames was proposed by the authors [3]. This paper focuses on a simplified approach for damage simulation and acceptance evaluation. The seismic demands of critical members are evaluated from the global demands predicted by RA and are compared with a detailed member based analysis. The member deformation capacity under cyclic loading is determined from the monotonic loading capacity by using a plastic skeleton shift damage model. Application of the proposed methods is studied for a six-storey two-bay frame for three ground motions and an acceleration impulse.

2. DEMAND EVALUATION BY REDUCED ANALYSIS

(1) OUTLINE OF REDUCED ANALYSIS [3]

A brief overview of a member hysteresis based analytical procedure and reduced analysis method is shown in Fig.1 and Fig. 2, respectively for understanding the simplifications in RA. In RA, a preliminary analysis based on first order reliability method and a simple equivalent-static model

of seismic loads is performed. The random equivalent-static force is given by a linear combination of deterministic modal force vectors and the random multiplier to each modal load profile, having zero-mean and standard deviation proportional to effective modal mass times the spectral acceleration. Using this model, the important collapse modes are judged from consideration of reliability index (β) and probabilistic network evaluation technique. Lower is the β , more important is the failure mode. The virtual work equations of only important collapse modes are used to arrange a global yield polyhedron model of the frame, which represents the elastic-perfectly-plastic idealisation of the global restoring force characteristics. In the inelastic state during dynamic analysis, the plastic components of deformation for the important collapse modes are computed by the normality rule on the yield polyhedron.

(2) LOCAL (MEMBER) PLASTIC DEFORMATION DEMAND

The local deformation demands are studied on a six-storey two-bay frame [4] as shown in Fig. 3. The columns and beams are composed of H-shaped and I-shaped steel members, respectively. The cross-section details are shown in Fig. 4 and Table 1. The storey height is 3.75m and span of each bay is 6m. The modulus of elasticity and yield stress for steel is taken as 205GPa and 235Mpa, respectively. The total weight of the frame is 3326.4KN and yield base shear coefficient is 0.11. The fundamental natural period is 2.57 seconds and the other details are given in reference [3]. The collapse modes under horizontal loads have been enumerated and are shown in Fig. 5. The important failure modes are listed in Table 2. The frame is analysed for El Centro (NS), 1940, Hachinohe (EW), 1968, Fukiai

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Fig. 1 Member Hysteresis Based Analytical Models (Deterioration and Simultaneous Acceptance Check)

Table 1 – Member Properties

Mem. ID	2b/h	θ_p (rad.) ($M_p L / 6EI$)	b/t _f	d/t _w	L/r _y
1	1.0	0.01018	6.15	16.75	92.68
2	1.0	0.00732	6.88	19.79	67.10
3	1.0	0.00809	6.67	18.89	74.05
4	1.0	0.00669	7.06	20.60	61.64
5	1.0	0.00615	7.43	22.50	56.85
6	0.5	0.00647	6.67	43.37	151.92
7	0.5	0.00718	6.69	41.83	158.41
8	0.5	0.00785	6.96	40.93	169.11
9	0.5	0.00859	7.01	39.24	179.07
10	0.5	0.01081	6.12	35.55	222.63

Fig. 2 Simplified Reduced Analysis Method for Damage Simulation and Acceptance Evaluation

Fig. 3 Six-Storey Two-Bay Frame

Fig. 5 Collapse Modes for six-storey two-bay frame

Fig. 4 Member Cross Section

Table 2 Important Failure Modes

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

Table 3- Details of Ground Motions

Ground Motion	Earthquake	PGA* (in 'g')	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

*Peak Ground Acceleration (in gravity acceleration)

(NS), 1995 ground motions and an impulse of 0.75 m/sec velocity. The details of the ground motions are shown in Table 3.

The plastic deformation history of each important collapse modes is computed in the response analysis. The plastic rotation history of the plastic hinges is calculated by the compatibility equation. If the plastic hinge belongs to some collapse modes, the summation of each plastic deformation history will be considered as the response history of the plastic hinge. As a matter of simplification, the collapse modes with cumulative plastic rotation, in one side, more than 1/200 radian are considered important for studying member deformation demand and these are listed in Table 4. As a further simplification, the members whose cumulative plastic rotation exceeds 0.01 radian are considered important to be studied for acceptance check. The value of 0.01 radian is selected considering that yield value of various member rotations (refer Table 1) range from 0.006 to 0.01 radians. In general, the members marked from C1 through C6 in Fig. 3 have undergone cumulative plastic rotations, in one side, in excess of 0.01 radians. The deformation demands of these members have been compared with a member based elastic-plastic analysis made using a computer program DIANA [5] to validate results of RA. The envelopes of cumulative plastic deformation and maximum plastic deformation obtained from RA and Diana have been plotted in Fig. 6 and 7, respectively. It can be seen that the demand prediction by RA is fairly good for practical purposes.

Table 4 Cumulative Plastic Rotation ($> 1/200$) of Important Failure Modes (RA)

Seismic Motion	Failure Mode ID	Positive Rotation $\sum \Delta \theta_{p,RA}^+$	Negative Rotation $\sum \Delta \theta_{p,RA}^-$
El-centro	19	0.0174	-0.0219
	14	0.0065	-0.0002
	1	0.0010	-0.0065
Hachinohe	14	0.0323	-0.0185
	19	0.0284	-0.0531
	5	0.0012	-0.0055
	2	0.0000	-0.0057
Fukiai	19	0.0306	-0.0956
	14	0.0015	-0.0119
	17	0.0110	0.0000
	1	0.0015	-0.0208
Impulse	19	0.0000	-0.0302
	14	0.0010	-0.0093
	1	0.0005	-0.0130

Figure 6 Envelopes of Cumulative Plastic Deformation

Figure 7 Envelopes of Maximum Plastic Deformation

3. DAMAGE SIMULATION BY SKELETON SHIFT MODEL

Damage may be defined as an unfavourable phenomenon/ state wherein a structure does not meet prescribed objective, which is usually related to non-restorable/ permanent plastic deformations that may adversely affect the performance.

The deformation demands in RA are computed for elastic-perfectly plastic idealisation of global restoring force characteristics i.e. without incorporation of deterioration/ degradation effects. Initiation of such effects is then simulated using a skeleton shift model on the deformation history and multiple offline acceptance checks are carried out. The objective of offline acceptance checks is only to detect the initiation of deterioration wherein deterioration characteristics are not so severe such as due to buckling, etc. The simplified analysis with offline acceptance checks (Fig. 2) is not suitable if the deterioration characteristics are severe such as due to brittle failure and/ or the sequence of deterioration/ yielding is to be traced. Under such conditions, detailed analytical models incorporating simultaneous acceptance checks (Fig. 1) should be used.

A brief description of the skeleton shift model and various damage simulations is provided in the following sections.

(1) SKELETON SHIFT MODEL

A skeleton shift model [6, 7, 8] was originally developed to simulate hysteretic behaviour of steel members focusing on non-stationary (or history dependent) change of plastic resistances. The model was considered to represent the uniaxial stress-strain hysteretic characteristics of steel under cyclic reversals. It was applied to axial force-deformation hysteretic characteristics of bar springs in multi-spring joint model, used to represent plastic zones in the beam-columns. Fig.8 shows the skeleton shift hysteresis rule for the bar springs. When plastic deformation along one side of the skeleton occurs, the target point in the loading side moves together with the loading point, and at the same time, the other side skeleton curve is shifted to the loading direction as much as ψ times the plastic deformation. Strictly, ψ should be a non-linear function of the deformation history. For simplicity and practical application, shifting of skeleton curve is taken as proportional to the movement along plastic skeleton, which is not likely to induce significant errors. The parameter ψ may range from 0 to 1 for steel

members. Fig. 9 and Fig. 10 compare the test and model results, for a box-section beam-column (width/thickness = 40), for monotonic loading and for El-Centro (NS), 1940 seismic wave (PGA = 0.35 g) [7]. Thus, this kind of hysteresis model including deterioration can be used for the simultaneous acceptance check shown in Fig. 1.

For energy absorption capacity evaluation, decomposition of hysteretic curve into the skeleton part and Bauschinger part has been proposed in reference [9] for round shape steel rods subjected to bending, wherein the skeleton part can be predicted by analytical methods and Bauschinger part can be estimated by empirical relationships. In our view, skeleton shift model is relatively simple, and design friendly, as it uses a single shift parameter to simulate both parts.

In this study, only the plastic component of the deformation is used for the damage simulation and to ascertain if member strength degradation below M_p occurred. The procedure is illustrated in Fig. 11.

(2) DETERIORATION-FREE AMPLITUDE IN LOCAL/ LATERAL BUCKLING FAILURE OF BEAM-COLUMNS

The shift parameter ψ can be calibrated from the deterioration observations in the constant plastic deflection amplitude cyclic loading tests. If ${}_p\theta_a$ is the constant plastic rotation amplitude (refer Fig. 12), then the cyclic cumulative skeleton plastic rotation, on each loading side, will form a geometric progression. Following relationship is obtained for N (≥ 1) integer cycles:

Negative side:

$${}_p\theta_a(1+\psi) + {}_p\theta_a(1+\psi)^2 + \dots + {}_p\theta_a(1+\psi)^{2(N-1)} = {}_p\theta_a \frac{(1-\psi^{2N})}{(1-\psi)}$$

Positive side, if $(1+\psi)^{2N-1} < 1$:

$${}_p\theta_a + {}_p\theta_a(1+\psi)^1 + \dots + {}_p\theta_a(1+\psi)^{2N-3} = {}_p\theta_a \frac{(1-\psi^{2N-1})}{(1-\psi)}$$

Positive side, if $(1+\psi)^{2N-1} > 1$:

$${}_p\theta_a + {}_p\theta_a(1+\psi)^1 + \dots + {}_p\theta_a(1+\psi)^{2N-1} - {}_p\theta_a = {}_p\theta_a \psi \frac{(1-\psi^{2N})}{(1-\psi)}$$

Fig. 8 Skeleton-Shift Hysteresis Rule for Bar Springs

Fig. 12 Constant plastic rotation Cyclic Hysteresis Loop

Fig. 9 Moment vs. Rotation Relationship After Meng, Ohi, Takanashi [7]

Fig. 11 Plastic Skeleton Shift Model for Damage Simulation

For $\psi \leq 1$, the maximum value of cumulative skeleton plastic rotation $\left(\sum_s {}_p\theta \right)$ will be on negative side i.e.

$$\sum_s {}_p\theta = {}_p\theta_a \frac{(1-\psi^{2N})}{(1-\psi)} \quad (1)$$

For large values of N , Eq. (1) can be approximated as:

$$\sum_s {}_p\theta \approx \frac{{}_p\theta_a}{(1-\psi)} \Rightarrow \psi \approx 1 - \frac{{}_p\theta_a}{\sum_s {}_p\theta} \quad (2)$$

Eq. (2) is used to calibrate ψ from the past experimental result [10] on H-shaped cross sections (SS-41, SM-50). In these tests, critical deflection amplitude in constant deflection amplitude cyclic loading were obtained, up to which load-deflection hysteresis loops are stable. Such an amplitude is termed as deterioration-free amplitude hereafter. Fig.13 shows one such hysteresis loop [10]. Table 5 lists critical plastic rotation capacity $[R_c = (\theta/\theta_p)_c - 1]$, normalised to yield rotation, for the tested members. Experimental studies [11] on similar cross-sections were carried out to determine the plastic rotation capacity under monotonic loading, with moment gradient, which can be idealised as shown in Fig. 14. The plastic rotation capacity corresponding to maximum moment ($M/M_p = \tau$) is shown by R_m in Table 5. The parameter ψ can be obtained as follows:

$$\psi \approx 1 - \frac{R_c}{R_m} \quad (3)$$

Figure 15 shows the ψ value v/s slenderness ratio (L/r_y) for steel grades SS-41 and SM-50. Within this data, ψ varies from about 0.5 to 0.8. The same range of ψ may hold good for n_y (ratio of applied axial load to yield axial load) ranging from 0.0 to 0.4 [7]. As a conservative measure for damage simulation, $\psi = 0.8$ is adopted. At higher order of n_y , ψ may be taken conservatively as 1.0. The grade of steel doesn't seem to influence ψ .

Fig. 10 Hysteresis Curve of Beam-Column (Shift Parameter $\psi = 0.8$) After Meng, Ohi, Takanashi [7]

Fig. 13 Hysteresis Loops, Test DG-130-4 After Udagawa, Takanashi, Tanaka [10]

(3) LOW-CYCLE FATIGUE

Idea of shift parameter ψ can also be used to simulate low cycle constant plastic strain fatigue characteristics, of the kind proposed by Coffin and Manson [12].

$$\varepsilon_p N^k = C \quad (4)$$

where, ε_p is the plastic strain amplitude (peak to peak) and k and C are material constants. The constant k is approximately taken as 0.5 and the constant C can be obtained by putting $\varepsilon_p/2 = \varepsilon_{l,m}$ for $N=1/4$ cycles, where $\varepsilon_{l,m}$ is the monotonic plastic strain capacity. Eq. (4) can be written as:

$$r_{c-m} N^{0.5} = 1 \quad (5)$$

where, $r_{c-m} = \varepsilon_p / \varepsilon_{l,m}$.

For the purpose of comparison, the ratio of constant cyclic plastic rotation to cumulative skeleton plastic rotation, in N cycles, can be obtained from Eq. (1) as:

$$r_s = 2p\theta_a / \sum_s p\theta = 2(1-\psi) / (1-\psi^{2N}) \quad (6)$$

Eqs. (5) and (6) have been calibrated such that for $N=1$ cycle:

$$r_{c-m} = r_s (\psi = 1) = 1 \quad (7)$$

Fig. 16 compares Eq. (6) and Eq. (7) for $N \geq 1$. However, for $N=1/4$ cycles r is shown as 2 for the sake of completeness. It appears that certain ψ values can simulate fatigue behaviour for a given range of r and N e.g. $\psi = 0.8$ apparently seem to simulate fatigue behaviour for $N \leq 10$. However, while combining fatigue acceptance with RA it should be noted that RA provides only global plastic deformation history and local partial yielding of the frame is ignored. Accordingly, ψ should be increased to compensate for the same to account for effects such as micro crack propagation, etc.

(4) OTHER SIMPLIFIED DAMAGE SIMULATIONS

It may be interesting to note that:

For $\psi = 0$, skeleton demand is not cumulated so far as deformation remains within the past peak values. This corresponds to acceptance evaluation by maximum deformation criteria. Performance of non-structural members may be checked against such criteria (Fig. 2).

For $\psi = 1$, each plastic deformation increment contributes to the skeleton demand. This corresponds to Kato-Akiyama model of acceptance evaluation by cumulative plastic deformation criteria (Fig. 2)

Table 5 Calibration of ψ value

Steel Grade	Mem.	Specimen ¹	L/r_y	R_c^1	R_m^2	ψ
SS-41	H 200x100x 5.5x8	DG-110-1	51.2	2.5	6.69	0.63
		DG-110-2	51.3	2.0	6.67	0.70
		DG-130-4	62.5	2.0	4.70	0.57
		DG-130-7	62.6	2.0	4.69	0.57
		DG-130-8	62.4	2.0	4.71	0.58
		DG-130-16	62.1	2.1	4.75	0.56
		DG-130-20	61.9	2.4	4.78	0.50
		DG-150-2	70.0	1.7	3.94	0.57
		DG-170-3	81.7	1.0	3.00	0.67
		DGC-120-2	50.8	3.3	14.6*	0.77
	H 100x100x 5.5x8	DGC-150-2	63.6	3.1	8.9*	0.65
		DGH-110-2	52.3	1.3	3.40	0.62
SM-50	H 100x100x 6x8	DGH-130-2	61.9	1.1	3.05	0.64
		DGH-150-2	71.6	1.0	2.25	0.56
		DGH-170-2	80.9	0.5	1.30	0.62

¹ See Reference [10] for details. * Higher value than shown is expected.

² R_m (DG series) = $5770x(L/r_y)^{1.71}$ [11]; R_m (DGC and DGH series) [11]

4. ACCEPTANCE CHECK

Load deformation relation under monotonic loading is evaluated as shown in Fig. 14 [13]. The strength ratio (τ) of a member is determined from the consideration of local buckling and flexural torsional buckling after Akiyama's empirical formulas. The ratio of rigidity in the strain hardening range to elastic rigidity is taken as 0.03 for simplicity. For the purpose of acceptance check, load-deformation curve is simplified and assumed to consist of elastic, perfectly plastic and degrading zones as shown in Fig. 14. In actual behaviour, degradation in member strength would begin after attaining maximum value M_m . In this paper, however, degradation is assumed when member strength falls below M_p and the cumulative plastic rotation capacity is limited to ($p\theta_{cr,m} = R_p\theta_p$). It may be noted that the definition of limiting value under monotonic loading ($p\theta_{cr,m}$) may not be unique and a different definition based on other considerations such as absorbed plastic energy, etc. may also be adopted. As a conservative measure, a member is considered acceptable if there is no initiation of member strength degradation (below M_p) i.e. the cyclic skeleton cumulative plastic rotation demand in one side ($\sum_{one,s} \Delta_p\theta$) is less than the monotonic plastic rotation capacity ($p\theta_{cr,m}$).

Skeleton Demand: $\sum_{one,s} \Delta_p\theta = \text{Max}(\sum_{+,s} \Delta_p\theta, -\sum_{-,s} \Delta_p\theta)$

Acceptance Check: $p\theta_{cr,m} \geq \sum_{one,s} \Delta_p\theta_{RA}$

Fig. 14 Load-Deformation Idealisation

Fig. 15 Calibration of ψ

Fig. 16 Fatigue Simulation

For $\psi = 0$, the skeleton demand ($\sum_{one,s} \Delta_p \theta$) will be same as the maximum plastic rotation demand. Table 6 compares the capacity and demand for critical members for $\psi = 0.8$ and $\psi = 1.0$. It is seen that the demand envelope is not sensitive to ψ values and $\psi = 1.0$ can be adopted in the present case. Insensitiveness of the demand envelope to ψ is due to the fact that there are only a few cycles of large plastic deformation in the chosen example. However, it should be noted that skeleton plastic deformation demand might be sensitive to selected ψ value, when there are many cycles of relatively small plastic deformations. Under such situations, adoption of higher ψ may lead to quite conservative scenario.

It can be seen from Table 6 that all the member are in the acceptable state, from the standpoint of structural safety itself, because the frame studied is composed of relatively stocky members. The global and local demands obtained from RA can also be used to judge the performance against the limiting values of interstorey drift angles, drift angle capacities of local connections and non-structural elements, etc indicated in the design codes [14,15]. For example, $\theta_{p \max}$ exceeds 1/15 radian to Kobe Earthquake (Fig. 7), which is not acceptable from the safety of non-structural elements e.g. curtain walls, etc.

Table 6 Cumulative plastic rotation capacity ($\rho \theta_{cr,m}$) and demand envelope ($\sum_{one} \Delta_p \theta_{RA}$)

Member ID	($\rho \theta_{cr,m}$)	$\psi = 0.8$ ($\sum_{one} \Delta_p \theta_{RA}$)	$\psi = 1.0$ ($\sum_{one} \Delta_p \theta_{RA}$)
C1	0.22866	0.0223	0.0225
C2	0.19959	0.0223	0.0225
C3	0.20218	0.1061	0.1075
C4	0.22095	0.1061	0.1075
C5	0.04837	0.0307	0.0336
C6	0.04961	0.0307	0.0336

5. CONCLUSIONS

A simple and design friendly, seismic acceptance evaluation procedure is proposed for multi-degree-of-freedom moment resisting frames. The application of proposed ideas is studied for a six-storey two-bay frame for El Centro (NS), Hachinohe (EW), Fukiai (NS) ground motions and an acceleration impulse. Following are the salient features:

1. Local (member) plastic deformations demand obtained from the simplified reduced analysis compare fairly well for practical purposes, with demand predicted by the detailed member based analysis procedures.
2. A plastic skeleton shift model is proposed for damage simulation. The skeleton shift parameter can be suitably adjusted for representing various kinds of damage phenomenon.
3. A comparison of skeleton cumulative cyclic plastic deformation demand with monotonic plastic deformation capacity is used to carry out a simple and conservative acceptance check.

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

著者らは、文献[3]において、鋼構造ラーメン骨組に対する終局地震荷重効果を、崩壊メカニズム数を重要なもののみに低減した降伏多面体モデルを用いた応答解析で評価する手法を提案した。そこでは荷重効果は、大局的な応答変位履歴ないし各崩壊機構形の塑性変形履歴の形で与えられる。本稿では、(1)この荷重効果に対して、鋼構造ラーメン骨組の性能が受容できるかどうかの判定を行う手法を示し、(2)鋼構造ラーメン部材の耐力劣化に関連した性能判定を、統一的に表現する累積損傷クライテリア、について提案する。

2. 簡略化応答解析による要求部材塑性変形性能

(1) 考慮する崩壊機構数を低減した簡略化応答解析 [3] の概要

予備解析として弾性地震荷重効果の静的水平荷重モデルとして、古典的規準モードの荷重基底ベクトルにランダムモード乗数を乗じて線形結合した荷重モデルを仮定する。モード乗数は零平均とし、標準偏差は、各次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。骨組の崩壊機構形の信頼性指標値を評価して小さいものを選択し、また相関の大きな複数の崩壊機構形は統合して、骨組の塑性崩壊に対する安全領域を降伏多面体モデルで近似する。安全領域の内部では弾性と仮定し、塑性変位増分の法線則によって応答解析を行う。

(2) 要求部材塑性変形性能の評価例

複数の崩壊機構形毎に塑性変形履歴が算定されるが、その塑性変形履歴から、適合式により崩壊機構形に属する塑性ヒンジの塑性回転角履歴を求める。塑性ヒンジが複数の崩壊機構形に属する場合は、それぞれの塑性回転角履歴の単純和を塑性ヒンジの応答履歴とみなす。文献[3]で例題とした6層2スパン骨組に4種の実地震波形を入力した結果、比較的大きな塑性変形を被る塑性ヒンジの累積塑性変形角ならびに最大塑性変形角について、簡略化応答解析結果（縦軸）と、塑性ヒンジ法による部材レベル弾塑性応答解析結果（横軸）とを図7に対照している。簡略化応答解析で概ね安全側に評価できることが分かる。

3. スケルトン移動型履歴モデルを利用した受容判定

上記の簡略化応答解析では、一般化された完全弾塑性挙動が仮定されているので、耐力劣化が顕著でない範囲の応答予測結果が得られる。その応答履歴を、耐力劣化現象も含むスケルトン移動型履歴モデルに適用して、耐力劣化が顕著になるかについての受容判定を行う。

(1) スケルトン移動型モデルの概要

歪み硬化、局部座屈・横座屈等の不安定現象による耐力劣化など、塑性抵抗力の変化を含む履歴モデルとして、第2著者の提案になるスケルトン移動型履歴モデル[6][7][8]を利用する。図8に履歴則を示しているが、1)正側、負側にスケルトン曲線を仮定し、それぞれに目標点を置く。初期位置としては塑性化開始点に置く。2) 載荷点が、一方のスケルトン曲線の塑性部分をたどると同時に、その上の目標点を載荷点と一緒に移動する。また塑性移動量の Ψ 倍（シフト係数）だけ、逆側のスケルトン曲線を同じ方向に移動する。3)一方のスケルトンから除荷した直後は、弾性除荷とするが、逆側のスケルトン上の目標点に向かう場合は、除荷点と目標点とを結ぶ軟化曲線上をたどる。

単調載荷曲線に適合するようにスケルトン曲線を定め、適切なシフト係数を仮定すると、耐力劣化の生じる鉄骨ラーメン部材の履歴曲線や地震応答性状を追跡できる。（図9、図10） スケルトン履歴型モデルを用いて部材レベルの応答解析を行えば、要求性能と性能の受容判定を同時に行えるが（図1）、本稿では要求性能については完全弾塑性

モデル、受容判定にのみスケルトン移動型モデルを用いる。（図2）

(2) 耐力劣化の発生しない繰返し変位振幅

スケルトン移動型履歴モデルでは、塑性抵抗力の変化は、各方向のスケルトン曲線上で経験する塑性変形の総和（スケルトン累積塑性変形）に依存することになるので、耐力劣化に関わる性能判定は、スケルトン累積塑性変形を参照する。また塑性変位一定振幅の繰返し載荷を行った場合、 N サイクル時のスケルトン累積塑性変形は(1)式で与えられ、 $0 < \Psi < 1$ のとき、 N を無限大の極限にすると(1)式は(2)式に収斂する。すなわち、十分小さな振幅で繰り返すと何度繰り返しても、スケルトン累積塑性変形はある値以上に進行しない。すなわちスケルトン移動型履歴則は、耐力劣化の発生しない限界変位振幅が存在することを内包している。宇田川らはH形断面部材について繰返し載荷実験を行い、図13のような限界振幅を観察している[10]。逆に、実験的に耐力劣化の生じない限界振幅ならびに単調載荷時の耐力劣化開始変位が測定されていれば、(3)式によってシフト係数 Ψ を同定することができる。宇田川らの実験[10][11]から同定した Ψ の値を表5、図15に示しているが、シフト係数 $\Psi = 0.5$ から 0.8 程度の値となる。

(3) Manson-Coffin 則との対応

低サイクル疲労破壊の場合に用いられる古典的な累積損傷則として、Manson-Coffin 則がある。(4)・(6)式、図16において、本稿におけるスケルトン移動型履歴モデルに基づく累積損傷則との比較を行っているが、ある塑性歪振幅、あるサイクル数の範囲内では、シフト係数を例えば 0.8 程度に設定して塑性歪履歴に適用すれば、式(4)のManson-Coffin 則と同等の累積損傷則を表現できるので、この種の低サイクル疲労のシミュレーションにも利用できる。しかしながら、1) Manson-Coffin 則の形からは、前項のような破壊の生じない塑性歪振幅は導出することはできないこと、2) 本稿で用いているような大局的応答の塑性履歴は、低サイクル疲労亀裂の成長に関連するような局所の塑性歪履歴を反映していないこと、に留意すべきである。

(4) その他の簡略化された損傷規範

- 1) $\Psi = 0$ とすると、過去に経験した最大応答変位を超過しない限り、繰返し塑性変形を受けてもスケルトン累積塑性変形が進行しない。すなわち、最大応答変位と単調変形能力の比較による性能判定になる。
- 2) $\Psi = 1$ とすると、ある方向の繰返し塑性変形増分が全てその方向のスケルトン累積塑性変形に算入されることになり、通常の累積塑性変形と単調変形能力の比較による加藤・秋山型の安全側に簡略化した性能判定になる。

4. 簡略化応答解析による性能評価例

2. の簡略化応答解析例で算定された塑性回転角履歴に対して、 $\Psi = 1$ ならびに 0.8 を用いて算定したスケルトン累積塑性回転角を表6に示しているが、ここで用いた地震応答履歴は繰返し回数が比較的小なかったため、両者に大きな差は認められなかった。

5. まとめ

鋼構造ラーメン骨組の崩壊機構数を重要なもののみに低減した降伏多面体モデルを用いて簡略化応答解析を行い、得られた崩壊機構毎の塑性変形履歴を、塑性ヒンジの塑性回転角履歴に変換する手法を示し、6層2スパン骨組の応答解析例によりその適用性を調べた。また得られた塑性回転角履歴をスケルトン移動型の履歴モデルに適用してスケルトン累積塑性変形を評価し、単調変形能力と比較することにより部材耐力劣化に関わる受容判定を簡便に行う手法を示した。

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