임의항복강도의 분포가 강구조물의 거동계수에 미치는 영향

Influence of the Random Yield Strength Distribution on the Behaviour Factor of Steel Structures

국 승 규[1)]

Kook, Seung Kyu

요 약 : 임의항복강도가 강구조물의 에너지소산능력에 미치는 영향을 파악하기 위해 본 논문에서는 7개의 강뼈대구조물을 모델링하여 응답스펙트럼해석법에 적용되는 거동계수를 산출하고 그 분포상태를 결정하였다. 또한 지진하중의 임의성이 거동계수에 미치는 영향과 비교하기 위해 주어진 스펙트럼을 만족하는 4개의 인공지진을 시뮬레이션하여 적용하였다. 본 연구의 특성상 방대한 양의 시간-이력계산을 수행하여야 하므로 근사해법인 시간-이력해석법을 개발하여 신뢰도를 검토하고 적용하였다.

ABSTRACT : In order to check the influence of the randomness in yield strengths on the energy dissipation capacity of steel structures, behaviour factors applied for the "Response Spectrum Method" and their distributions are determined in this study with 7 steel framed models. Also 4 artificial accelerograms simulated with a given spectrum are applied to check the influence of the randomness in seismic action on the behviour factor. To execute numerous time-step calculations for the investigation a time-step analysis method is developed and applied after the reliability estimation to determine the action effects.

핵 심 용 어 : 임의항복강도, 강구조물의 에너지소산능력, 응답스펙트럼해석법, 거동계수, 지진하중의 임의성, 시간-이력해석법

KEYWORDS : randomness in yield strength, energy dissipation capacity of steel structures, response spectrum method, behaviour factor, randomness in seismic inputs, time-step analysis method

1. Introduction

Steel structures might have same members with different yield strengths due to the random characteristics in the yield stress as well as in the section modulus. Under the seismic

1) 정회원, 부경대학교 토목공학과 교수

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action this fact implies a significant change of the structural behaviour in the plastic range because of the activation as well as the deactivation of possible plastic hinges. The random yield strength distribution over steel structures should be therefore considered as a regularity condition for materials. For example as a requirement for the strength distribution over a steel framed structure in order to avoid an unexpected overstrength, the following condition

$$\max . r_i - \min . r_i \leq 0.2 \qquad (1)$$

where r_i is the ratio of f_{yr_i}/f_{yi}, f_{yr_i} is the actual yield strength of a member i and f_{yi} is the design yield strength of the same member i, is given in Eurocode 8, Part 1-3.[1]

With the structural regularity conditions the seismic design is normally performed using the "Response Spectrum Method", where the non-linear behaviour of structures is taken into account by the behaviour factors. The seismic forces calculated using an elastic model for a structure are reduced with this factor to more realistic values.

The study objective is defined, in this regard, to investigate the influence of the randomness in yield strengths on the energy dissipation capacity provided by the nonlinear system behaviour and the influence is evaluated as a distribution of the behaviour factors.

Also the seismic action should be considered as a random factor in the seismic design. Therefore, the influence of random yield strengths on the distribution of behaviour factors is compared with the influence of randomness in seismic inputs by applying 4 artificial accelerograms simulated with a given spectrum.

2. Preparation of Input Data

2.1 Random yield strengths

For the preparation of random yield strengths as input data a real strength distribution provided from the quality control project carried out by Sedlacek[2] is selected. The distribution function of the selected real strength distribution is given in Fig. 1, where M and $M_{pl,nom}$ denote real yield strength values and the nominal yield strength value respectively.

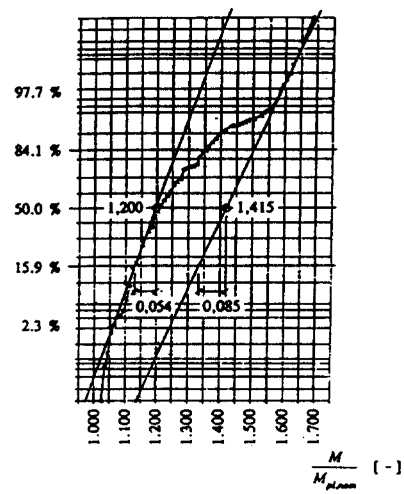

Fig. 1 Real strength distribution

The selected strength distribution is approximated as a mixed distribution of two normal distributions and the mean values and standard deviations are estimated for each normal distribution. For the generation of random yield strengths the "Monte-Carlo-Method"[3] is adopted. An example of the generation results is given in Fig. 2. In order to get a satisfied approximation to the selected real strength distribution 5000 random values are generated. The comparison of the two assumed normal distribution functions shows that the generation is sufficiently carried out for the investigation.

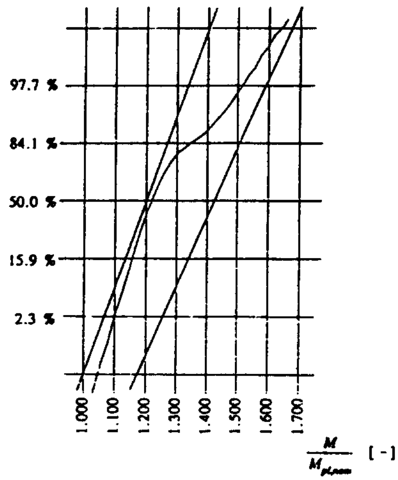

Fig. 2 Example of the generation results(HEA 240)

2.2 Seismic inputs

For the preparation of seismic inputs 4 artificial accelerograms are simulated, which have a time-dependent envelope function with a strong phase duration of 6 seconds and satisfy a spectrum used for structures in nuclear power plants.[4] One of the simulated accelerograms is given in Fig. 3.

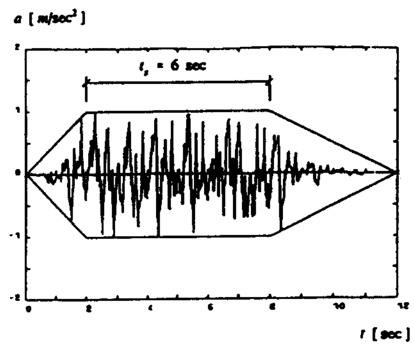

Fig. 3 Artificial accelerogram (Acc. 1)

2.3 Modelling of the investigated systems

For this study 7 steel framed systems, 4 one-storey and 3 multi-storey frames, are modelled, where HEA 240 and IPE 240 sections (Table 1) are selected for columns and beams respectively. For the modelled systems given in Fig. 4 possible plastic hinges are specified at column bases and at both ends of beams.

Table 1. Section properties (HEA 240 & IPE 240)

member	h mm	b mm	t_w mm	t_f mm	A cm^2	Z_{pl} cm^3	$M_{pl,nom}$ kNm
HEA240 (column)	230	240	9.5	12.0	76.8	744.0	174.84
IPE240 (beam)	240	120	6.2	9.8	39.1	366.0	86.01

note : h=height ; b=width ; t_w=web thickness
t_f=flange thickness ; A=section area
Z_{pl}=plastic section modulus
$M_{pl,nom}$=nominal yield strength

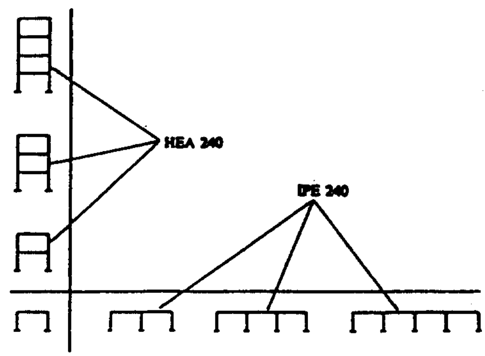

Fig. 4 Investigated systems

Such a modelling is done following the "Weak Beam Strong Column" concept according to the "Capacity Design" required by the seismic design and should be justified by checking the column overstrength condition, so-called "Column-Overdesign-Factor (COF)"

$$COF_{\min.} = \frac{5\%\ fraction\ value\ (col.)}{95\%\ fraction\ value\ (beam)} \quad (2)$$

which is determined from the strength ratio of the two elements. While a min. required factor of 1.2 is determined by Kato,[5] the min. factor of $COF_{\min.} = 1.48$ has been obtained from the generation results of the random yield strength distributions, which justifies the modelled systems. For each system 200 systems having different yield strengths at the possible plastic hinges are simulated, where the yield strengths are again randomly selected from the two input data sets for columns and beams.

3. Analysis Method

3.1 Development of a time-step analysis method

In order to carry out numerous computations of the dynamic system responses in the non-linear range it is not economically justifiable to apply a conventional time-step analysis method. Therefore a time-step analysis method based on the "Dynamic Plastic Hinge Method"[6] is developed such that the static system is controlled and modified after each time-step Δt according to the activation or deactivation of the specified plastic hinges (Fig. 5). The time-dependent variables $\ddot{u}_g(t)$, $s(t)$ and $u(t)$ denote earthquake excitation, section forces and displacements respectively.

The developed time-step analysis method uses the modal analysis technique, where the nonlinear behaviour is described with the plastic hinge theory. The modal characteristics of all detected systems are stored and reused within the program and the transformation of modal amplitudes between the modified system and the previous system is carried out. Either the fundamental mode only (Simplified Method) or all modes from the system degree of freedom (Multimodal Method) can be considered by the method for the determination of the dynamic system responses.

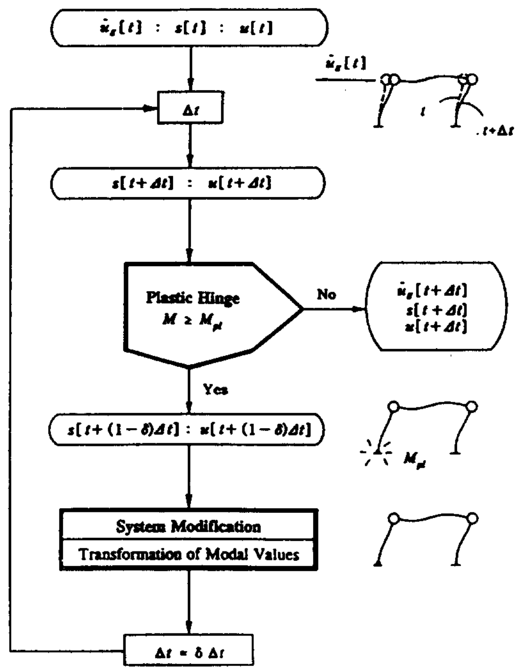

Fig. 5 Time-step of the analysis method

3.2 Reliability estimation

In order to check the reliability of the developed analysis method comparison calculations are executed. As an exact Finite-Element-Method the FE program DYNACS[7] (Dynamic Analysis of Composite and Steel Structures) is adopted. The representative comparison results are given in Fig. 6 and Fig. 7, where the displacement response δ is obtained at the top floor of the modelled systems.

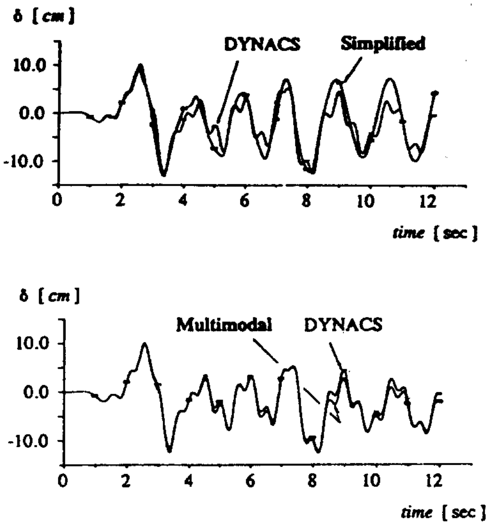

Fig. 6 Comparison calculation in the elastic range(4 storey 1 bay frame, Acc. 1)

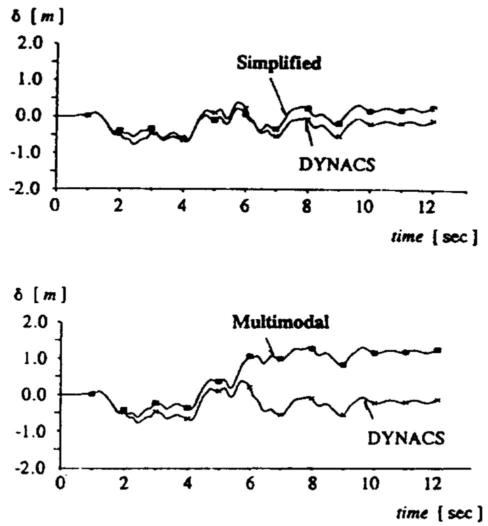

Fig. 7 Comparison calculation in the elastic-plestic range(2 storey 1 bay frame, Acc. 2)

When the system remains within the elastic range, i.e. under low acceleration the first plastic hinge is not yet formed, the Multimodal Method provides practically the same results as those of DYNACS (Fig. 6). The Simplified Method provides small differences in results from those of DYNACS, because the effects of higher modes are neglected. On the other hand, if the system is in the elastic-plastic range, i.e. under high acceleration with the formation of plastic hinges, the Simplified Method provides more accurate results than the Multimodal Method (Fig. 7). This is due to the plastic hinges specified for the investigated systems, the responses of which in the plastic range are governed by the fundamental mode. For the investigation in this study it is decided to use the Simplified Method providing sufficient accuracy.

4. Determination of the Behaviour Factor

4.1 "Procedure of Ballio / Perotti"

For the determination of the behaviour factor the "Procedure of Ballio/Perotti"[(8)] is applied, where the behaviour factor q is determined through the comparison of the displacement response of a real elastic-plastic system with that of an assumed hyperelastic system (Fig. 8) given by

$$q = \frac{a_p}{a_e} = \frac{\delta_p}{\delta_e} \tag{3}$$

in which a_e, δ_e are the acceleration factor and the displacement response at the elastic limit state and a_p, δ_p are those at the plastic limit state.

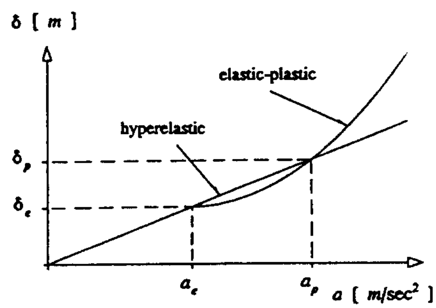

Fig. 8 Procedure of Ballio/Perotti

4.2 Presentation of the calculation results

An example of the statistic evaluation of the obtained behaviour factors is presented in Fig. 9. The behaviour factors determined for the 200 random systems and for a system with nominal yield strengths $M_{pl,nom}$ are given as a histogram and then simpler presentations are made, where the vertical line represents the scatter range of one standard deviation $\pm\sigma$ from the mean value m indicated by the horizontal line. The dotted line shows where the behaviour factor q_{nom} of the system with nominal yield strengths lies. The scatters of the obtained behaviour factors due to the random yield strengths as well as due to the different accelerograms are presented in Fig. 10 for all investigated systems. It is shown that the distribution of behaviour factors depends rather strongly on the random seismic input, different accelerograms, than on the random yield strength distribution.

5. Conclusions

In this study the influence of the random yield strength distribution over steel framed

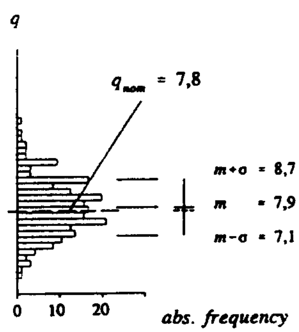

Fig. 9 Example of the statistic evaluation(1 storey 3 bay frame, Acc. 1)

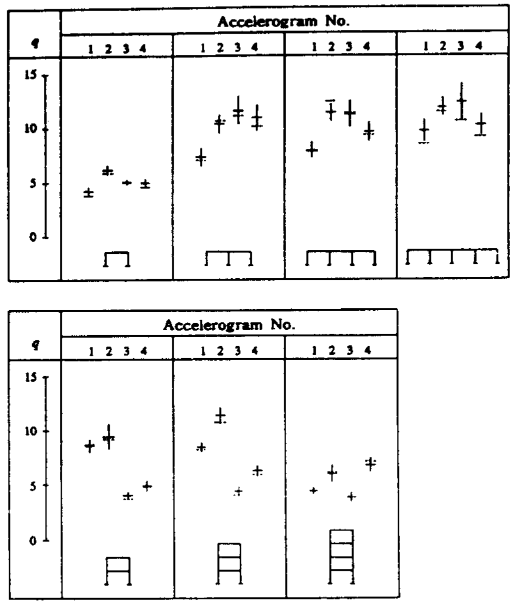

Fig. 10 Scatters of the obtained behaviour factors

yield strength distribution over steel framed structures on the nonlinear behaviour under the seismic action is investigated and evaluated as the distribution of behaviour factors. Also 4 artificial accelerograms are applied in order to compare the influence of the randomness in seismic action with that of the randomness in yield strengths. The investigated systems are modelled with the "Weak Beam Strong Column" concept according to the "Capacity Design" required by the seismic design. From the study results following conclusions are obtained with regard to the determination of behaviour factors of steel framed structures:

◆ The random yield strength distribution need not be considered, when the "Weak Beam Strong Column" concept is satisfied for steel framed structures.

◆ The influence of the randomness in seismic action should be considered. Therefore sufficient number of accelerograms should be applied (Further researches are needed to decide the required number of seismic inputs.).

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