Earthquake Resistant Design of a Steel Framed Structure in Low Seismic Regions Based on the Dynamic Behaviour

동적거동에 기초한 약진지역 철골뼈대구조물의 내진설계

국 승 규*
Kook, Seung Kyu

국문요약

구조물 내진설계의 개념은 기본요구조건이라는 조항으로 시방서에 규정되어 있으며 구조물이 지진발생시에 안전성과 경제성을 최대한 확보할 수 있는 파괴메카니즘을 갖추도록 설계할 것을 요구하고 있다. 요구되는 파괴메카니즘은 설계단계에서 구조물 형식에 따라 적합한 내진설계방식을 도입하여 확보할 수 있으며 비선형시간이력해석을 수행하여 지진시의 동적거동을 기술함으로써 확인할 수 있다. 내진설계에 보편적으로 적용하는 응답스펙트럼해석법은 선형해석법으로 구조물의 비선형동적거동의 영향을 거동계수로 반영하므로 파괴메카니즘 및 기본요구조건의 만족여부는 거동계수를 구하는 과정으로 결정할 수 있다. 이 연구에서는 내진설계방식에 의해 설계된 약진지역에 위치한 화학공장건물의 모델인 3차원 철골뼈대구조물을 선정하고 거동계수를 결정하는 과정을 수행하여 지진시의 동적거동을 확인하였다. 이 연구의 결과, 현 시방서의 응답스펙트럼해석법에 적용되는 거동계수는 강진지역의 구조물의 경우 기능성 및 안정성 한계를 제시하지만 약진지역 구조물의 경우는 실제 동적거동과 무관하다는 것과 약진지역에 위치한 구조물의 내진설계에는 시방서가 제시한 내진설계방식을 적용하는 것이 주요한 사항임을 확인하였다.

주요어 : 내진설계, 파괴메카니즘, 비선형시간이력해석, 동적거동, 거동계수, 약진지역, 3차원 철골뼈대구조물

ABSTRACT

The concept of the earthquake resistant design for structures is set forth as the basic requirements in the codes, which require failure mechanisms under earthquakes in view of safety and economy. The required failure mechanisms can be obtained by introducing the earthquake resistant design methods according to the structural types at the design stage and confirmed by describing the dynamic behaviour under earthquakes through non-linear time step analyses. As the response spectrum method, generally applied as a linear analysis method, reflects the effects of the non-linear dynamic behaviour of structures by introducing behaviour factors, the fulfillment of the basic requirements as well as the failure mechanisms can be verified through the determination procedure of the behaviour factors. In this study, a spatial steel framed structure, a model for a chemical industry building located in area classified as low seismic regions, is selected which is designed on the basis of the earthquake resistant design methods. The dynamic behaviour under earthquakes is investigated by way of the determination procedure of the behaviour factors. Based on the study results, it is confirmed that the behaviour factors applied by the response spectrum method have nothing to do with the actual dynamic behaviour of structures in low seismic regions, while they provide both the serviceability limit and the ultimate limit for structures in strong seismic regions. Also, it is confirmed that proper application of the earthquake resistant design methods provided in the codes is essential for the earthquake resistant design of structures in low seismic regions.

Key words : earthquake resistant design, failure mechanisms, non-linear time step analyses, dynamic behaviour, behaviour factor, low seismic regions, spatial steel framed structure

1. Introduction

The concept of the earthquake resistant design for structures is defined in the codes[(1),(2)] as the following basic requirements.

- Minimization of damage under earthquakes with high probability of occurrence during the design life; serviceability limit state
- No-collapse requirement under the design seismic event; ultimate limit state

The design concept requires safe and economic failure mechanisms for structures under earthquakes. Such failure mechanisms should be ductile, which can be obtained at the design stage by introducing the earthquake resistant design methods. In case of steel framed structures different methods are provided according to the structural types. The fulfillment of the basic requirements as well as the ductile failure mechanisms is to be checked with detailed data about linear as well as non-linear dynamic behaviour of structures under earthquakes. To this end the non-linear time step analyses, somewhat rigorous, are to be carried out, where the selected seismic inputs should represent the characteristics of the regional seismicity exactly enough in the statistical sense.

In Eurocode 8[(1)], the behaviour factors provided for various structural types are applied by the response spectrum analysis method which is codified as a simplified linear

* 정회원 · 부경대학교 건설공학부, 조교수(대표저자 : skkook@pknu.ac.kr)
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analysis method. The behaviour factors reflecting the non-linear structural behaviour under earthquakes are determined for structures based on the above mentioned earthquake resistant design methods. The procedures of the response spectrum method provided in the present codes are originally prepared for structures in strong seismic regions. The ultimate limit state is to the structural behaviour under the design seismic event and the serviceability limit state is to the required elastic limit determined by applying the specified behaviour factors. Generally structures designed with the non-seismic load combinations should be strengthened to satisfy the required elastic limit. In this regard the two limit states of a structure under earthquakes can be verified through the determination procedure of the behaviour factor. However it is not the case for structures in low seismic regions because the structural behaviour can be within the linear range even under the design seismic event. Therefore the application of the response spectrum method as well as the interpretation of the results should be reviewed compared with the actual dynamic behaviour under earthquakes in view of the design concept, on which the earthquake resistant design for structures in low seismic regions should be based.

In this study, a spatial steel framed structure model consisted with two plane frame types is selected for a chemical industry building located in area classified as low seismic regions. The two plane frames are designed such that they have the ductile failure mechanisms and satisfy the non-seismic load combinations. The non-linear time step calculations are carried out with the program DYNACS[3] to investigate the dynamic behaviour under earthquakes and the behaviour factors are determined from the calculation results. For seismic inputs 10 synthetic motions are simulated, which reflect the characteristics of recorded earthquakes in the vicinity of the site.

2. Analysis model

The spatial steel framed structure is given in Fig. 1 together with the global coordinates, where two plane frame types are arranged as follows.

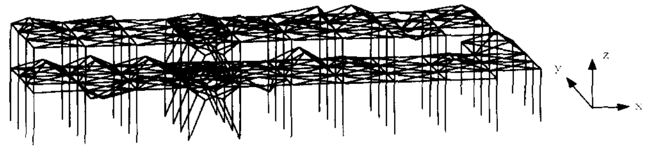

Fig. 1 Model of the spatial steel framed structure with the global coordinates

- 4 concentric braced frames(CBF) in the global x direction with a 5m distance
- 10 moment resisting frames(MRF) in the global y direction with a 6m distance

The concentric braced frames are 2-storey 9-bay frames and the moment resisting frames are 2-storey 3-bay frames. The spatial steel framed structure has an 1-storey entrance attachment neglected in the two plane frame types. The element properties are given in Table 1, where the strength values are calculated with the nominal values. HE 400B profiles are used for both plane frame types as column elements with the strong member axis in the global x direction. As beam elements IPE 500 and IPE 550 profiles are used for the concentric braced frames and for the moment resisting frames respectively. The two plane frame types are designed with the non-seismic load combinations and also with the ductile failure mechanisms according to the earthquake resistant design methods. For the concentric braced frames the concentric braces are designed as the dissipative elements and for the moment resisting frames the "weak beam-strong column" concept is introduced.

The modal analysis of the spatial steel framed structure provides that the first two modes given in Fig. 2 and Fig. 3 are the horizontal modes in the global x and y direction respectively. The frequencies as well as the mode shapes of the two modes are taken as references for the modeling of the two plane frames.

Table 1 Element properties of the spatial steel framed structure(w : weak axis, s : strong axis)

Element	G [kgf/m]	h [mm]	b [mm]	t_w [mm]	t_f [mm]	A [cm²]	N_{pl} [kN]	W_{pl} [cm³]	M_{pl} [kNm]	Q_{pl} [KN]
HE 400B(w)	155.3	400	300	13.5	24	197.8	4747.2	1104.0	265.0	1995.3
IPE 500	90.7	500	200	10.2	16	115.5	2772.5	2194.1	526.6	661.4
Brace	11.1	L 100×50×10				14.1	338.4	-	-	-
HE 400B(s)	155.3	400	300	13.5	24	197.8	4747.2	3231.7	775.6	658.5
IPE 550	105.5	550	210	11.1	17.2	134.4	3226.1	2787.0	668.9	792.4

G : weight per meter　h : cross-sectional height　b : flange width　t_w : web thickness
t_f : flange thickness　A : cross-sectional area　N_{pl} : plastic axial force　W_{pl} : plastic section modulus
M_{pl} : plastic moment　Q_{pl} : plastic shear force

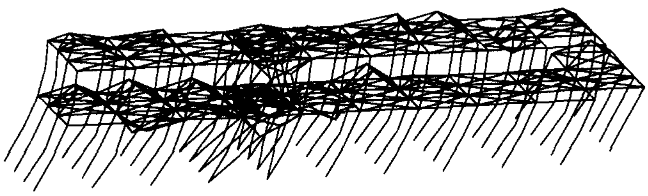

Fig. 2 First mode of the spatial steel framed structure model (horizontal mode in the global x direction, ω=4.16rad/s)

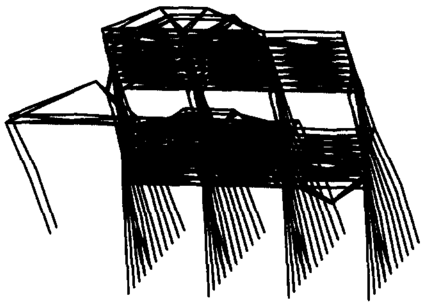

Fig. 3 Second mode of the spatial steel framed structure model (horizontal mode in the global y direction, ω=9.74 rad/s)

The analysis model for the concentric braced frame and the first mode are shown in Fig. 4. Besides the element self-weight the floor load of the spatial steel framed structure is converted into a distributed mass of 1.25ton/m and included in the beam elements. The lumped masses,

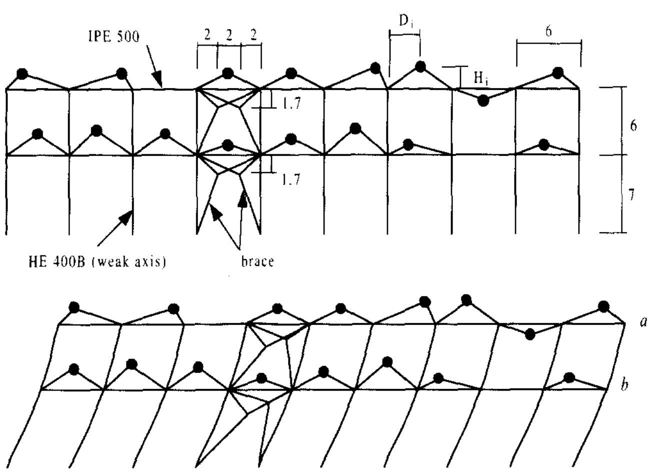

Fig. 4 Analysis model for the concentric braced frame and the first mode with ω=3.82rad/s (unit in m)

equipments of the chemical industry but not shown in Fig. 1 for the sake of brevity, are taken from the most loaded frame among 10 concentric braced frames. Weightless rigid elements are used to locate the lumped masses in their planned positions given in Table 2 in order to include the P-Δ effect. The frequency of the analysis model is compared with the corresponding frequency of the spatial steel framed structure and the ratio of 0.92 is accepted for proper modeling.

The analysis model for the moment resisting frame and the first mode are shown in Fig. 5. The floor load of the spatial steel framed structure is converted into a distributed mass of 1.50ton/m. The lumped masses are modeled in the same way as those for the concentric braced frame and their planned positions are given in Table 3. The frequency ratio of 0.98 between the analysis model and the corresponding frequency of the spatial steel framed structure is accepted.

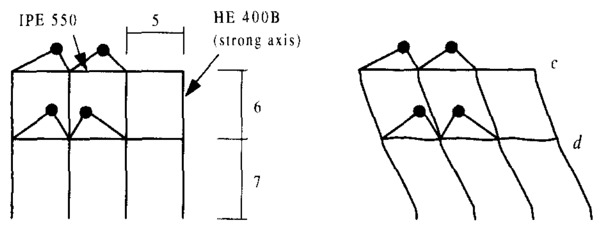

Fig. 5 Analysis model for the moment resisting frame and the first mode with ω=9.50rad/s (unit in m)

Table 3 Lumped masses of the analysis model for the moment resisting frame

Bay no.		1	2	3
2nd floor	Mass[ton]	10.5	21.3	-
	Hi[m]	2.0	2.0	
	Di[m]	4.0	3.0	
1st floor	Mass[ton]	8.5	9.5	-
	Hi[m]	2.5	2.5	
	Di[m]	3.5	1.5	

Table 2 Lumped masses of the analysis model for the concentric braced frame

Bay no.		1	2	3	4	5	6	7	8	9
2nd floor	Mass[ton]	7.0	10.0	-	15.9	10.7	24.5	23.0	30.0	15.0
	Hi[m]	1.5	1.5	-	1.5	1.5	2.0	2.0	-1.0	1.5
	Di[m]	1.5	5.0	-	3.0	3.0	5.0	3.0	3.0	4.0
1st floor	Mass[ton]	7.5	22.0	7.0	5.1	2.3	18.0	1.3	-	2.5
	Hi[m]	2.0	2.3	2.0	1.0	1.5	2.5	1.0	-	1.0
	Di[m]	3.0	2.5	3.0	3.0	3.0	3.0	2.0	-	2.5

H_i : height of the lumped mass from the floor

D_i : distance of the lumped mass from the nearest left column

3. Synthetic motions

With the program SIMQKE[4] 10 synthetic motions, acceleration time histories, are simulated as seismic inputs, for which the elastic response spectrum provided in DIN 4149[5], NAD(national application document) of Eurocode 8 is taken as target response spectrum. The parameters applied for the given soil type B3 provided in DIN 4149 are listed in Table 4. S is the soil parameter, η is the damping correction factor (η =1 for 5% viscous damping) and β_0 is the spectral acceleration amplification factor for 5% viscous damping. T_B & T_C are the limits of the constant spectral acceleration branch and T_D is the value defining the beginning of the constant displacement range. k_1 & k_2 are exponents which influence the shape of the spectrum for a vibration period greater than T_C & T_D respectively.

The design ground acceleration a_g of 0.6m/s^2 is taken as an effective peak ground acceleration(EPA) of the design seismic event. The synthetic motions have a total duration of 16 seconds with 2 seconds rise time followed by 5 seconds level time. The intensity and duration of the synthetic motions are determined with recorded earthquakes in the vicinity of the site classified as low seismic regions. Fig. 6 shows one of the simulated motions, which is used as seismic input 01. Both the target elastic response spectrum and the response spectrum of the synthetic motion 01 are presented in Fig. 7. The simulated synthetic motions are different in their frequency contents and power spectra, which represents the random characteristic of earthquakes.

Table 4 Parameters for the elastic response spectrum(soil type B3)

S	η	β_0	T_B[s]	T_C[s]	T_D[s]	k_1	k_2
1.0	1.0	2.5	0.1	0.6	2.0	1.0	2.0

4. Behaviour factors for the plane frames

4.1 Determination procedure

The behaviour factor is determined according to the definition of Ballio/Perotti[6] which is graphically shown in Fig. 8. The displacements of the reference node, e.g. top of SDOF model, obtained with stepwise increased intensities of a seismic input constitute a non-linear curve, which is compared with the extrapolated linear elastic behaviour. The curve is normalized by the acceleration value a_y leading to the elastic limit of the structure, the coordinate (1,1) in Fig. 8. The elastic limit, factor 1 value, means the first occurrence of the yielding in the structural member. According to the definition of Ballio/Perotti the behaviour factor is obtained at the intersection point, q_a=q_d, and this point is regarded as the dynamic stability limit, which means the formation of the failure mechanism. The q factor as defined in Eurocode 8, Part 1-1[7] indicates the energy dissipation capacity of a structure through mainly ductile behaviour of its elements.

The response spectrum method provided in the present codes adopts the q factor to avoid the non-linear analysis. The dynamic stability limit corresponds to the ultimate limit state under the design seismic event and the q-reduced limit, the required elastic limit, corresponds to the serviceability limit state. However this interpretation is applicable only for structures in strong seismic regions as intended in the codes, because the two limit states are determined within the range of non-linear structural behaviour. Also it should be noted that the behaviour factors provided in the codes are applicable only for structures based on the earthquake resistant design methods.

In the following calculations the effects of probable expected strengths(overstrengths), post-elastic stiffness as

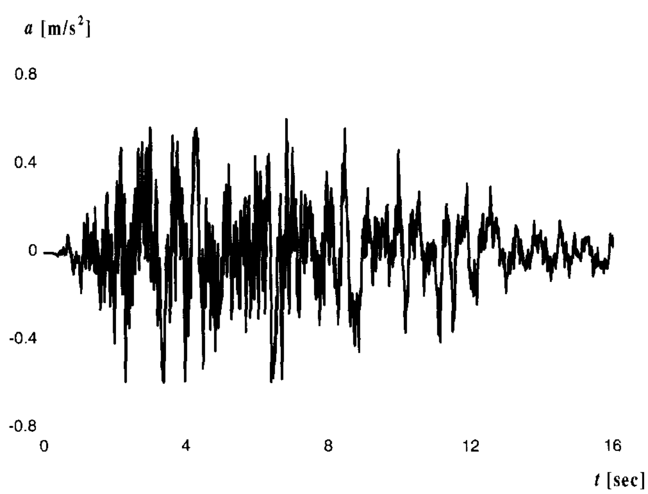

Fig. 6 Synthetic motion 01(seismic input 01, a_g of 0.6m/sec^2)

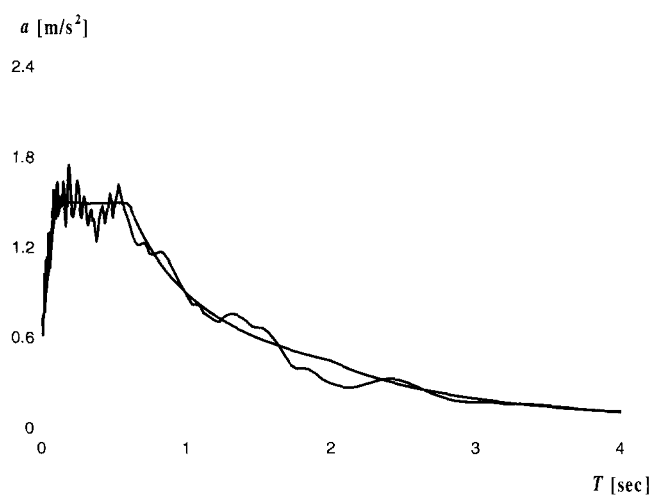

Fig. 7 Response spectrum of the synthetic motion 01(5% damping)

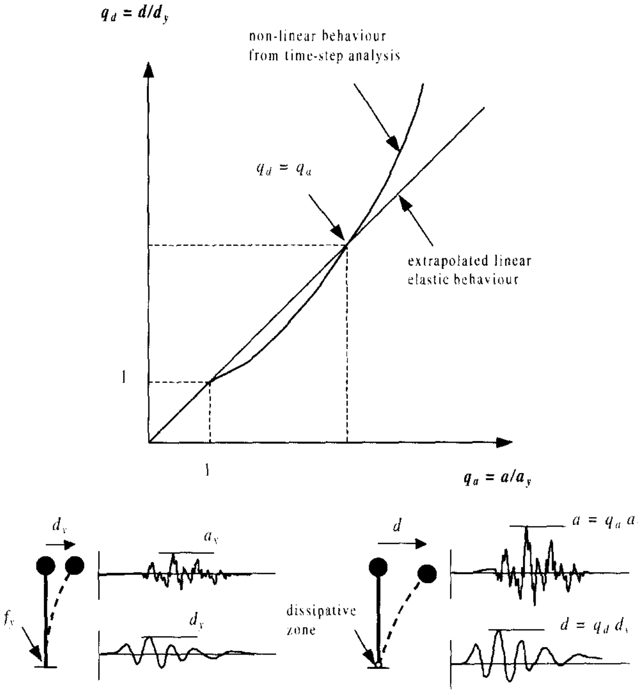

Fig. 8 Definition of the behaviour factor

well as probable dissipative behaviour of the beam-column panel joint are not included, because such effects are relatively small compared to the influence of the random characteristic inherent in earthquakes on the calculated behaviour factors. Increased behaviour factors can be obtained by considering these effects.

4.2 Concentric braced frame(CBF)

4.2.1 Non-linear behaviour

The determination of the behaviour factor requires the estimation of the elastic limit for the first step. For the concentric braced frame the dissipative elements are the concentric braces. From the non-linear axial force-displacement characteristic of the longest brace obtained through a displacement controlled calculation, N - u diagram given in Fig. 9, the elastic limit of 321kN is estimated for the analysis model. The actual compressive strength of the brace should be greater than the calculated maximum compression of 7.6kN. This is due to the analysis model for the brace, where a hinge is assumed at the centre of the brace for the buckling load. It is assumed that such a model provides analysis results on the safe side.

Fig. 10 shows a force time history of a concentric brace with max. tension of 187kN obtained with the seismic input 01, a_g of 0.6m/sec^2. From the force time histories of all concentric braces acceleration values leading to the elastic limit are estimated for each seismic input and given as factor 1 values in Table 5.

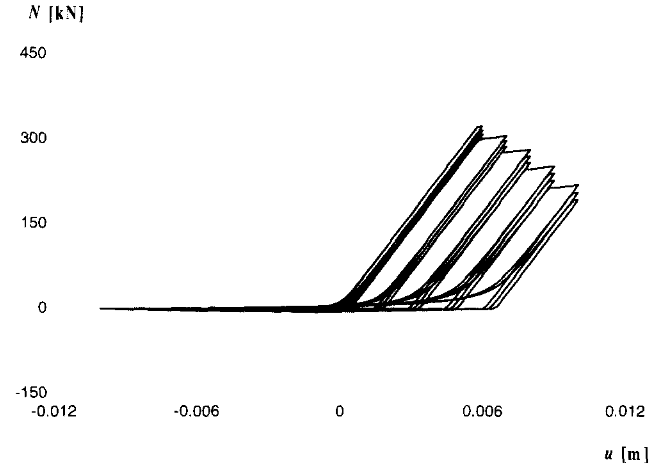

Fig. 9 Axial force-displacement characteristic of the longest brace (N_{max} = 321.0kN, N_{min} = -7.6kN)

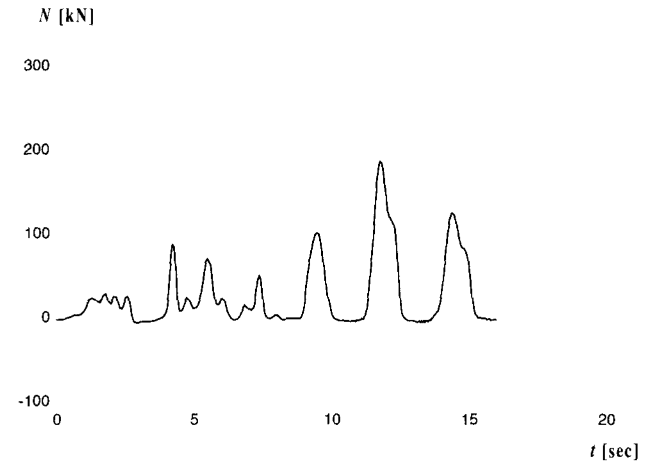

Fig. 10 Force time history of a brace with max. tension obtained with the seismic input 01, a_g of 0.6m/sec^2(N_{max} = 187.0kN, N_{min} = -6.7kN)

Table 5 Factor 1 values for the concentric braced frame(normalized by a_g of 0.6m/sec^2)

Seismic input	01	02	03	04	05	06	07	08	09	10
Factor 1	1.69	1.60	1.40	1.67	1.20	1.77	1.40	1.40	1.38	1.56

The second step is to calculate the displacement time histories of the reference nodes a & b shown in Fig. 4. Starting with the factor 1 values and subsequently with the multiples of the factor 1 values, the absolute maximum displacements are obtained from the displacement time histories as shown in Fig. 11, where the factor 3 means an acceleration value of 3.04(=0.6×1.69×3)m/sec^2.

4.2.2 Behaviour factor

The calculated q factors for the concentric braced frame are listed in Table 6. The most conservative factor q of 2.0 obtained with the seismic input 01 shown in Fig. 12 should be decided as the behaviour factor for the concentric braced frame. The conservative decision is based on the following facts. On the one hand the random characteristic of earthquakes should be considered on the safe side. On the other hand the behaviour factors are calculated with the model designed according to the earthquake resistant design methods, in which the stable non-linear hysteresis is assumed. The stable hysteresis can be taken only when the connections of the concentric braces are designed according to the capacity design considering material overstrengths in order to prevent unintended connection failures. The behaviour factor of 2.0 corresponds to the provided q factor of 2 for this frame type in Eurocode 8, Part 1-3.[8]

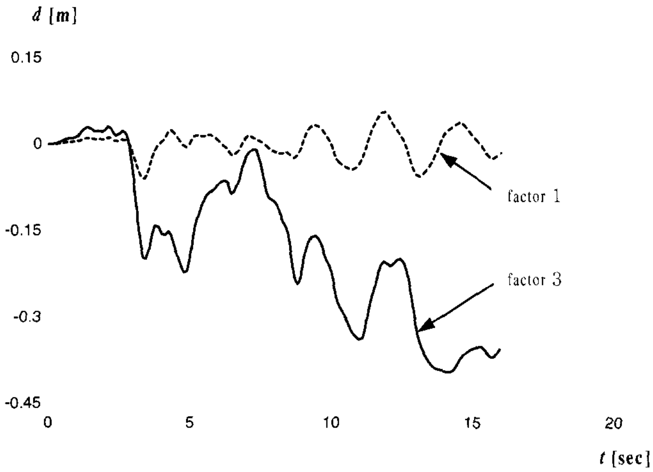

Fig. 11 Displacement time histories of the reference node a obtained with the seismic input 01(factor 1 : d_{min} = −0.060m, factor 3 : d_{min} = −0.394m)

Table 6 Calculated q factors for the concentric braced frame

Seismic input	01	02	03	04	05	06	07	08	09	10
q	2.0	3.0	4.0	2.0	2.0	3.0	3.0	4.0	2.0	5.0

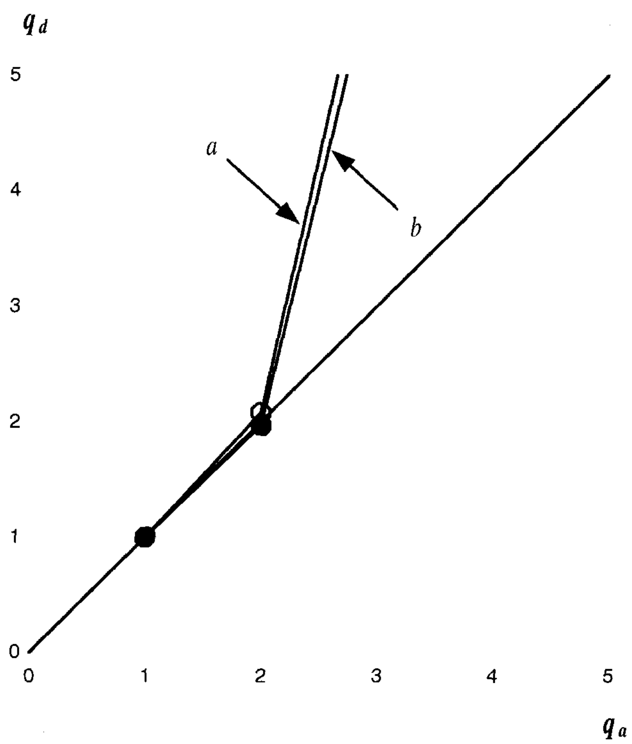

Fig. 12 Results of the non-linear calculations obtained with the seismic input 01

4.3 Moment resisting frame(MRF)

4.3.1 Non-linear behaviour

For the moment resisting frame the non-linear behaviour of the elements are restricted at the beam ends as well as at the bases of columns according to the "weak beam-strong column" concept. Fig. 13 shows a moment-rotation hysteresis, M - φ diagram, of a column base obtained with the seismic input 01, factor 5, which means an acceleration value of $11.91(=0.6\times3.97\times5)\mathrm{m/sec^2}$. The elastic limit for the moment resisting frame is computed with the tool indicating plastification ratio provided by the program DYNACS.

The same procedures are carried out for the moment resisting frame as those for the concentric braced frame. Factor 1 values given in Table 7 are determined for the first step. Then the displacement time histories of the reference nodes, c & d in Fig. 5, are calculated with the factor 1 values as well as with the multiples of the factor 1 values as shown in Fig. 14, from which the absolute maximum displacements are obtained.

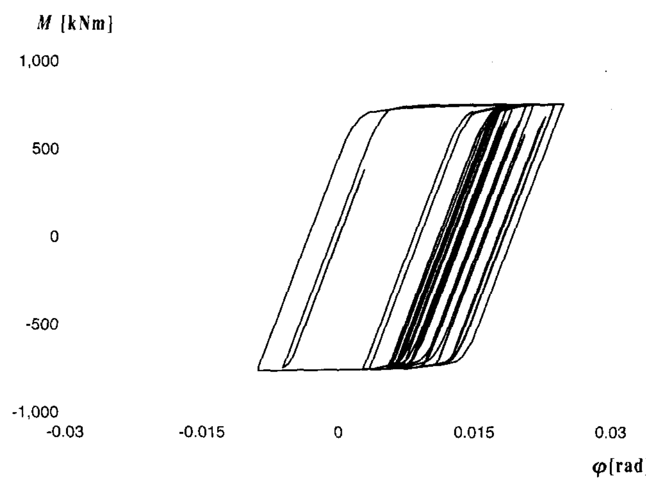

Fig. 13 Hysteresis of a column base obtained with the seismic input 01, factor 5(M_{min} = −762kNm, φ_{min} = −0.0089, M_{max} = 761kNm, φ_{max} = 0.0233)

Table 7 Factor 1 values for the moment resisting frame(normalized by a_g of 0.6m/sec^2)

Seismic input	01	02	03	04	05	06	07	08	09	10
Factor 1	3.97	2.73	2.48	3.22	3.49	3.21	2.41	2.94	2.94	2.93

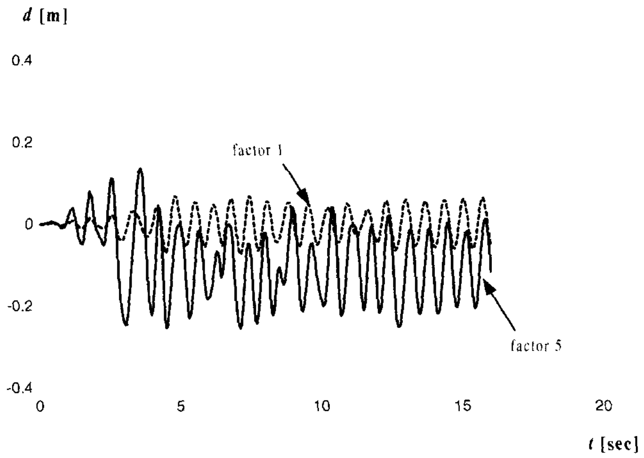

Fig. 14 Displacement time histories of the reference node c obtained with the seismic input 01(factor 1 : d_{min} = -0.074m, factor 5 : d_{min} = -0.257m)

4.3.2 Behaviour factor

The calculated q factors for the moment resisting frame are listed in Table 8. The most conservative factor q of 5.5 obtained with the seismic input 09 shown in Fig. 15 should be decided as the behaviour factor for the moment resisting frame. The conservative decision is based on the same facts as those for the concentric braced frame. The behaviour factor of 5.0~6.0 is also provided for this frame type in Eurocode 8, Part 1-3.[8]

Table 8 Calculated q factors for the moment resisting frame

Seismic input	01	02	03	04	05	06	07	08	09	10
q	7.5	>10	>10	>10	6.0	6.0	>10	>10	5.5	>10

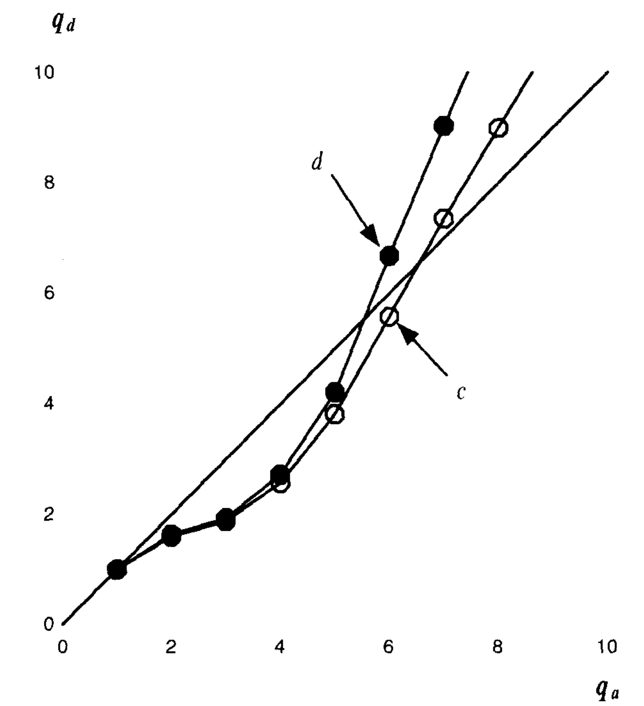

Fig. 15 Results of the non-linear calculations obtained with the seismic input 09

5. Two limit states of the spatial steel framed structure

For the verification of fulfillment of the basic requirements set forth in the codes the acceleration values leading to the elastic limit and the dynamic stability limit should be decided. From the calculation results the seismic input numbers and the minimum acceleration values leading to the two limits of the plane frames are given in Table 9. Because of the random characteristic of earthquakes in their acting direction it should be determined that for the selected spatial steel framed structure the two limits are governed by the dynamic behaviour of the concentric braced frame.

- elastic limit(serviceability limit state) : 0.72m/s^2
- dynamic stability limit(ultimate limit state) : 2.03m/s^2

It is confirmed that the elastic limit of the concentric braced frame is higher than the design seismic event, a_g of 0.6m/s^2. Obviously the structural behaviour is within the linear range under the design seismic event and it is not necessary to decide the required elastic limit with the specified behaviour factor.

Table 9 Acceleration values leading to the two limits of the two plane frames

Frame	Elastic limit		Dynamic stability limit	
	Seismic input	a_g[m/sec^2]	Seismic input	a_g[m/sec^2]
CBF	05	0.72	01	2.03
MRF	07	1.45	09	9.70

6. Conclusions

In this study a spatial steel framed structure, a model for a chemical industry building located in area classified as low seismic regions, is selected, which includes 4 concentric braced frames and 10 moment resisting frames. The two plane frame types are designed on the basis of the earthquake resistant design methods leading to the ductile failure mechanisms and they satisfy the non-seismic load combinations. The dynamic behaviour of the two plane frame models under earthquakes is investigated by way of the determination procedure of the behaviour factor. The ultimate limit state as well as the serviceability limit state are decided for the selected structure. From the study results conceptual procedures are summarized as below for the earthquake resistant design of structures in low seismic regions.

Step 1 : At the design stage the earthquake resistant design methods provided in the codes should be applied to obtain the ductile failure mechanism. The energy dissipation capacity is ensured in this way and the behaviour factors provided in the codes are applicable. It is sufficient to satisfy the non-seismic load combinations.

Step 2 : The response spectrum method without adopting the behaviour factor is applied to check the structural behaviour under the design seismic event.

case i) linear behaviour under the design seismic event; It is not necessary to start redesign and reanalyses. Without procedure of step 1 it is not sufficient to confirm only that the structural behaviour under the design seismic event is within the linear range, because such structures might experience brittle failures under possible big events.

case ii) non-linear behaviour under the design seismic event; The required elastic limit is to be calculated by applying the corresponding behaviour factor and the structural behaviour is checked for the required elastic limit. If the actual elastic limit is higher than the required elastic limit, redesign and new analysis are not necessary. If the actual elastic limit is lower than the required elastic limit, which is generally the case for structures in strong seismic regions, redesign should be carried out to satisfy the required elastic limit and reanalyses are necessary.

Step 3 : Detailed verification of the earthquake resistant design methods such as material overstrengths, connections, beam-column panel joints should be carried out in order to ensure the intended ductile failure mechanism.

Acknowledgement

This study is supported by the Pukyong National University, the Korea Earthquake Engineering Research Center and the Steel Construction Institute of RWTH Aachen.

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