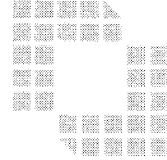


Korean Building Code 2016 (KBC 2016)

2016 Korean Building Code
Chapter 3 Design Loads

1. Principles of earthquake design

1.1 Major factors affecting magnitude of earthquake load

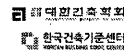


September 2016

Earthquake load = inertia force

Inertia force = the function of

- 1) peak ground acceleration,
- 2) structure mass, and
- 3) resonance(when the structure period is close to that of ground motion)



지진력은 지반의 동적인 운동에 의하여 구조물에 발생하는 관성력이다. 관성력은 구조물의 질량과 가속도의 곱으로 정의된다. 따라서, 구조물에 발생하는 관성력은 크게 세가지 변수의 영향을 받는데, 그 지역에 발생하는 지반의 최대가속도의 크기, 구조물의 질량, 그리고 구조물과 지반의 상호작용이다. 구조물의 질량은 용도, 시스템, 재료에 따라서 결정되며, 지반의 최대가속도 크기는 그 지역의 현재와 과거의 지반활동에 따라서 결정된다. 반면에 구조물과 지반의 상호작용은 구조물의 진동주기와 지반의 진동주기의 상호관계에 따라서 결정된다. 구조물의 진동주기가 지반의 진동주기와 비슷하면 공진현상이 나타나서 지반의 가속도보다 증폭된 가속도가 구조물에 발생하여 구조물에 발생하는 지진력(관성력)을 증가시킨다. 반면에 구조물의 진동주기가 지반의 진동주기와 크게 다르면 구조물에 발생하는 가속도의 크기가 감소하여 심지어 지반의 가속도보다도 작아질 수 있으며, 이로 인하여 지진력이 감소한다.

1.2 Response spectrum

Spectral accelerations (PSa) of structures can be conveniently estimated by using response spectrum. The response spectrum defines probable maximum accelerations of structures, as the function of structural period. Thus, without performing time history analysis, the design earthquake load ($= M \times PSa$) can be conveniently calculated.

이러한 구조물의 주기의 변화에 따른 구조물에 발생하는 최대 가속도의 변화를 나타낸 그림을 응답스펙트럼이라고 한다. 현행기준에서는 이 응답스펙트럼을 규정하여 동적해석없이 응

답스펙트럼값을 사용하여 지진하중을 쉽게 결정할 수 있도록 하고 있다. 건물의 주기를 평가하고 이 건물주기에 해당하는 응답스펙트럼값(가속도)을 결정한 후 이 값에 건물의 총 질량을 곱하여 건물에 작용하는 총 지진력을 산출한다. 이 총 지진력(밀면전단력)을 건물높이에 대하여 분배하여 지진하중의 분포를 결정한다.

1.3 effect of soft soil

As the stiffness of soil decreases, the response spectrum values (spectral accelerations) increases. For this, the soil factors are defined as the function of the soil stiffness.

내진설계기준은 과거의 지진이력과 그로 인한 피해에 대한 경험을 근거로 하여 발전이 되어 왔다. 그 주요한 사례가 연약지반의 영향과 건물의 연성능력이다. 1985 년 발생한 멕시코지진에서는 연약지반으로 인하여 많은 고층건물이 지진에 의하여 붕괴되었는데, 이후 내진설계 기준에서는 연약지반에 대한 영향을 강화하여 연약지반이 있는 경우에는 지진하중을 크게 증폭하였다. 연약지반의 주기는 암반의 주기보다 길어지는데, 이에 따라서 지반과 공진하는 건물의 주기도 길어져서 보다 긴 주기를 갖는 고층건물에 지진하중이 커진다. 후속연구와 지반계측결과에 의하면 고층건물뿐만 아니라 저층 건물의 경우에도 연약지반에 의하여 지진력이 크게 증폭되는 것으로 알려졌다, 현행기준에서는 이러한 연약지반에 대한 증폭계수가 고려되어 있다.

1.4 effect of ductility

Under rare and strong earthquakes, it is almost impossible to economically design structures, limiting severe damage. Thus, modern design codes permit to consider the ductility capacity of structures for the earthquake design. It is well known that as the ductility capacity increases, the design earthquake load can be reduced.

In KBC, the effect of ductility is addressed using the response modification factor (R factor). The R factor is defined according to the types of structural system and materials which are the major factor determining the structural ductility.

어느 나라든 내진설계에 사용되는 극한지진은 최소 500 년 이상의 재현주기를 갖는 지진을 사용하고 있다. 최대 100 년정도에 불과한 일반건축물의 수명과 비교할 때 이러한 극한지진 하중은 건축물의 사용기간 동안 발생하지 않을 확률이 훨씬 높다.

이때, 이러한 최대지진에 대하여 구조물이 별 손상없이 거의 탄성상태로 하중을 지지하려고 설계한다면 현재 구조물의 설계보다 매우 큰 강도로 설계되어야 하며, 따라서, 이러한 탄성

설계는 경제적으로나 건축공간적으로 수용하기 어렵다.

그러나 앞서 언급한 바와 같이 최대지진은 건축물의 사용기간 동안 발생하지 않을 확률이 크기 때문에 최대지진의 발생시에는 건물의 보존보다는 인명피해의 최소화에 중점을 두는 것이 적절한 내진설계전략으로 받아들여지고 있다. 이러한 관점에서 보면 굳이 건물의 탄성 거동에 의하여 발생하는 최대탄성지진력에 대하여 건물을 설계할 필요가 없다. 그 이유는 지진력이란 하중의 크기가 결정되어 있는 하중이 아니고 지반과 건물의 상호작용에 의하여 발생하는 관성력이기 때문에 그 상호작용을 조절하면 건물에 발생하는 지진력을 감소시킬 수 있다. 이러한 관점을 잘 반영할 수 있는 설계가 연성설계이다. 연성능력이란 한 구조물이 하중재하능력의 저하없이 비탄성변형을 일으킬 수 있는 능력을 말하는데, 건물의 구조시스템과 각 부재가 연성능력을 가지고 있다면, 지진발생시 비탄성거동이 일어나면서 구조물의 주기를 증가시켜서 공진현상을 줄이고 에너지 소산을 증가시켜서 지반의 움직임의 영향을 최소화 할 수 있다.

이러한 구조물의 연성능력을 반영하기 위하여 현행설계기준에서는 구조물의 연성능력에 반 비례하여 지진하중을 감소시킬 수 있도록 허용하고 있다. 구조물의 연성능력을 반영하는 계수를 반응수정계수라고 정의한다. 철골모멘트골조와 같이 연성능력이 큰 구조물은 작은 지진하중에 대하여 설계할 수 있도록 허용하고 있으며, 콘크리트와 같이 연성재료가 아닌 경우에는 연성능력을 확보하도록 철근연성상세를 엄격히 규정하고 있다.

1.5 ductility details

To satisfy the target ductility that is used to determine the earthquake load, the relevant ductility details should be satisfied. In concrete structures, confinement hoops should be closely spaced to prevent brittle failure of concrete in beams and columns, and shear failure should be prevented. In steel structures, weld and bolt connections between steel members should be carefully designed to prevent brittle failure. In particular, mechanical devices such as damper and base-isolators can be used to significantly enhance the ductility and energy dissipation capacity of the structures.

일반적으로 수용이 가능한 규모, 강도, 공사비로 건설된 대부분의 건축물은 최대지진발생시에 비탄성거동을 나타내며, 따라서 현행기준에서는 건축물이 일정 정도의 연성능력을 확보하여 저감된 지진하중에 대하여 설계하도록 요구하고 있다. 따라서 건축물의 연성능력 확보를 위한 정확한 내진상세가 콘크리트, 강구조 설계기준에 규정되어 있다.

콘크리트구조에 대해서는 소성한지가 예상되는 기둥과 보 단부의 횡보강과 전단파괴를 지연하기 위한 성능기반전단설계등이 규정되어 있으며, 강구조에서는 기둥-보 접합부등 연성에

취약한 접합부에 대한 상세가 규정되어 있다.

내진상세의 준수여부에 따라서 구조물의 실제 내진성능이 결정되므로 설계시에 내진상세를 반드시 준수하여야 한다.

2. Analytical / design Methods

The types of analysis methods which are used for earthquake design is determined according to the seismic design category of the building. As the importance of the building and peak ground acceleration increase, the design category varies from A to D. For the high category such as C and D, more accurate and sophisticate analysis methods such as dynamic analysis should be used to enhance the safety of the structure. For the general use and low rise buildings, equivalent static procedure is sufficient.

Types of numerical analysis

Static analysis

Linear static analysis

Nonlinear static analysis (push-over analysis)

Time history (time domain) dynamic analysis

Linear analysis direct integration

Modal superposition

Nonlinear analysis direct integration

Frequency domain analysis (linear analysis)

Fourier Transform

Modal analysis using response spectrum

Code-specified analysis

Equivalent static analysis (linear static analysis)

Dynamic analysis (Modal analysis using response spectrum)

For performance-based design, nonlinear static or nonlinear time history analysis should

be used to verify the deformation capacity of structures and members

2.1 Equivalent static procedure (등가정적 해석)

- linear analysis
- nonlinear analysis (no detailed specifications on nonlinear analysis)

Generally, the equivalent static method is applied to design category A and B, which are of ordinary use, assuming the dynamic mode and period of the structures.

Design procedure

- 1) Estimate the dynamic period of building using empirical equations. As the building height increases, the period increases.
- 2) With the estimated period, spectral acceleration of the structure is taken from the given response spectrum.
- 3) By multiplying the mass of the structure with the spectral acceleration, the base shear (total earthquake load) is calculated.
- 4) Assuming the dynamic mode of the structure, the base shear is distributed from the top to the bottom of the building.
- 5) For the given story loads, linear analysis is performed for the structure, and member design is performed to satisfy the force demand.

2.2 dynamic analysis (동적 해석)

- Modal superposition method using design response spectrum – main method
- Linear time history analysis
- Nonlinear time history analysis (no specific comments)

For buildings in Design category C : Table 0306.4.5

Regular buildings of more than 70m height or more than 21 stories

Irregular buildings of more than 20m or more than 6 stories

For buildings in Design category D : Table 0306.4.6

동적해석방법에는 시간이력해석과 모드해석법이 있는데, 시간이력해석은 지진의 시간이력별

로 구조물의 이력을 계산하며, 모드해석법에서는 다자유도의 구조물의 응답을 단 자유도로 분리하여 단자유도의 응답을 계산한 후 이 응답들을 중첩하여 전체 구조물에 대한 응답을 계산한다. 기준에서는 모드해석법을 제시하고 있는데, 그 이유는 모드해석법에서는 단자유도의 응답을 구할때 기준에서 제시하는 응답스펙트럼을 사용하여 쉽게 그 최대응답을 구할 수 있어서 쉽게 실무에 적용할 수 있기 때문이다. 상대적으로 시간이력해석에서는 신뢰성있는 다양한 지진이력에 대한 확보의 어려움과 해석의 복잡성 등 실무설계에 적용하기에 단점이 있다.

동적해석법은 주로 내진설계범주 C, D에 해당하는 구조물의 해석과 설계에 사용한다

2.3 nonlinear analysis (비선형해석)

Under strong earthquakes, majority of buildings are expected to experience large inelastic deformations and severe damage. Thus, it is reasonable to use nonlinear analysis for earthquake design of structures. However, in current design codes, for convenience in design and to avoid technical difficulties, equivalent static procedure based on linear elastic analysis and reduced design response spectrum is used.

However, recently, advanced nonlinear analysis and software have been developed. Furthermore, performance-based design based on nonlinear analysis will be popular in near future. Thus, the importance of nonlinear analysis is increasing.

최대지진발생시에는 구조물이 비탄성거동을 하므로 실제적으로는 정적 또는 동적 비탄성 해석을 수행하여야 한다. 그러나, 실무설계의 편의를 위하여 현행설계기준에서는 탄성해석을 기반으로 규정되어 있기 때문에 비탄성해석에 대해서는 특별한 언급이 없다. 그러나, 구조 해석과 설계기술이 발전함에 따라서 많은 정적비탄성해석과 설계방법이 개발되었으며, 동적 비탄성해석도 수행되고 있어서 향후 점차적으로 구조설계나 안전성평가에 비탄성해석의 적용이 증가할 것으로 예상된다.

3. Regional factor

Korea is divided into two earthquake regions. Generally, major regions and cities with high populations belong to region 1.

In region 1, the peak ground acceleration of 2400 year return period EQ is defined as 0.22g. In addition, the peak ground acceleration can be determined from the hazard map

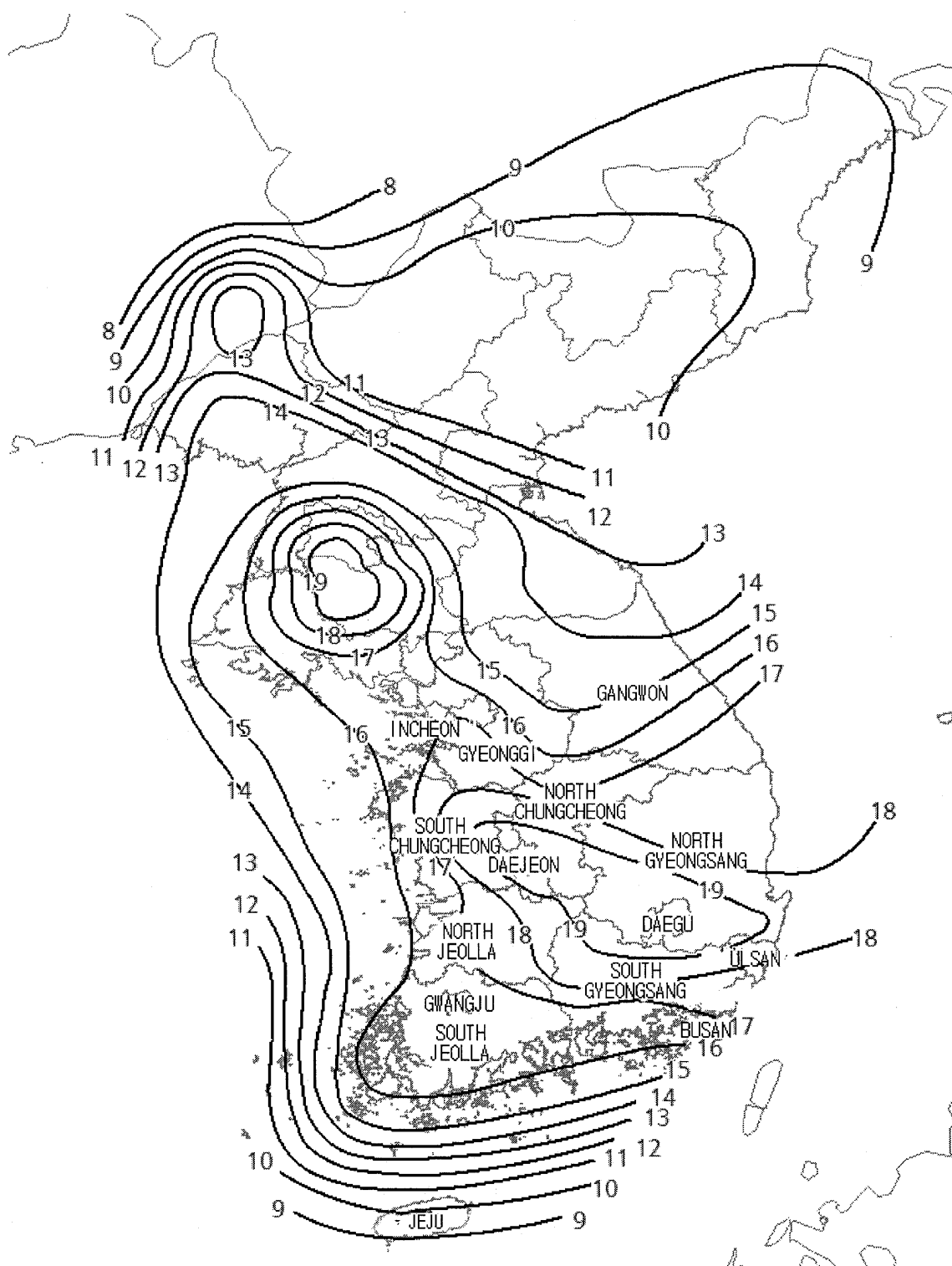
of Korea. However, the minimum value should be greater than 80 % of the acceleration determined from the Table, considering the uncertainty of the hazard map

<Table 0306.3.1> REGION OF SEISMICITY AND SPECTRAL ACCELERATION PARAMETER

Region of seismicity	Administrative district		Effective ground acceleration parameters
1	City	Seoul, Incheon, Daejeon, Busan, Daegu, Ulsan, Gwangju, Sejong	0.22g
	District	Gyeonggi-do, Chungcheongbuk-do, Chungcheongnam-do, Gyeongsangbuk-do, Gyeongsangnam-do, Jeollabuk-do, Jeollanam-do, South Gangwon-do*	
2	District	North Gangwon-do**, Jeju-do	0.14g

* South Gangwon-do: Yeongwol, Jeongseon, Samcheok, Gangneung, Donghae, Wonju, Taebaek

** North Gangwon-do: Hongcheon, Cheolwon, Hwhacheon, Hoengseong, Pyeongchang, Yanggu, Inje, Goseong, Yangyang, Chuncheon, Sokcho



[Fig. 0306.3.1] SEISMIC HAZARD MAP, EFFECTIVE GROUND ACCELERATION(S, %) OF MAXIMUM CONSIDERED EARTHQUAKE WITH A 2400-YEAR MEAN RECURRENCE INTERVAL; (NATIONAL EMERGENCY MANAGEMENT AGENCY, 2013)

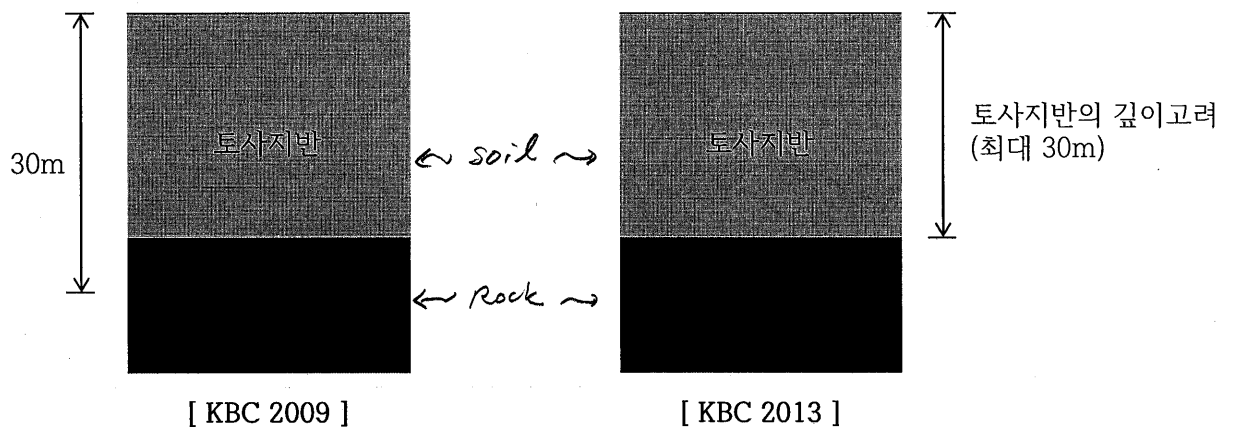
4. Site classification

The site is classified based on the average soil properties of the soil from the ground level to the ordinary rock (shear wave velocity = 760m/s). When the location of the ordinary rock is less than 5m or more than 30m, the average soil property is estimated for the top 30m soil. The site is classified from S_A (hard rock) to S_E (soft soil).

Ground motion is mainly affected by shear waves (or surface wave). Thus, site is classified based on the shear modulus

<Table 0306.3.2> SITE CLASSIFICATION

Site class	Soil profile name	Average site properties		
		Shear wave velocity (m/s)	Standard penetration resistance, \bar{N} (blows/300mm)	Soil undrained shear strength, \bar{s}_u ($\times 10^{-3}$ MPa)
S_A	Hard rock	>1500	-	-
S_B	Rock	760 to 1500		
S_C	Very dense soil or soft rock	360 to 760	> 50	> 100
S_D	Stiff soil	180 to 360	15 to 50	50 to 100
S_E	Soft soil	<180	< 15	< 50



5. Design response spectrum

S_{DS} = short period spectral acceleration (constant acceleration)

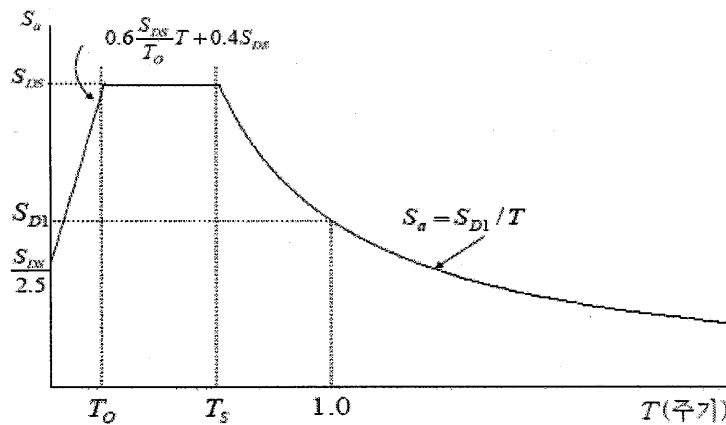
S_{D1} = 1 second spectral acceleration (constant velocity)

$$S_a = 0.6 \frac{S_{DS}}{T_o} T + 0.4 S_{DS}$$

$$S_a = \frac{S_{D1}}{T}$$

$$T_o = 0.2 S_{D1} / S_{DS}$$

$$T_s = S_{D1} / S_{DS}$$



Elastic design response spectrum

$$S_{DS} = S(2.5)F_a(2/3)$$

$$S_{D1} = SF_v(2/3)$$

F_a = short period site coefficient (Table 0306.3.3)

F_v = long period site coefficient (Table 0306.3.45)

S = effective peak ground acceleration corresponding to collapse limit state

(2/3) = reduction factor considering probability of structure collapse

(2.5) = amplification factor for short period structures (constant acceleration)

$S(2/3)$ = effective peak ground acceleration corresponding to Life safety limit state

6. soil coefficient

As the soil become soft, the spectral accelerations are amplified. Such effects are included in the soil coefficient.

<Table 0306.3.3> SITE COEFFICIENT FOR SHORT PERIODS, F_a

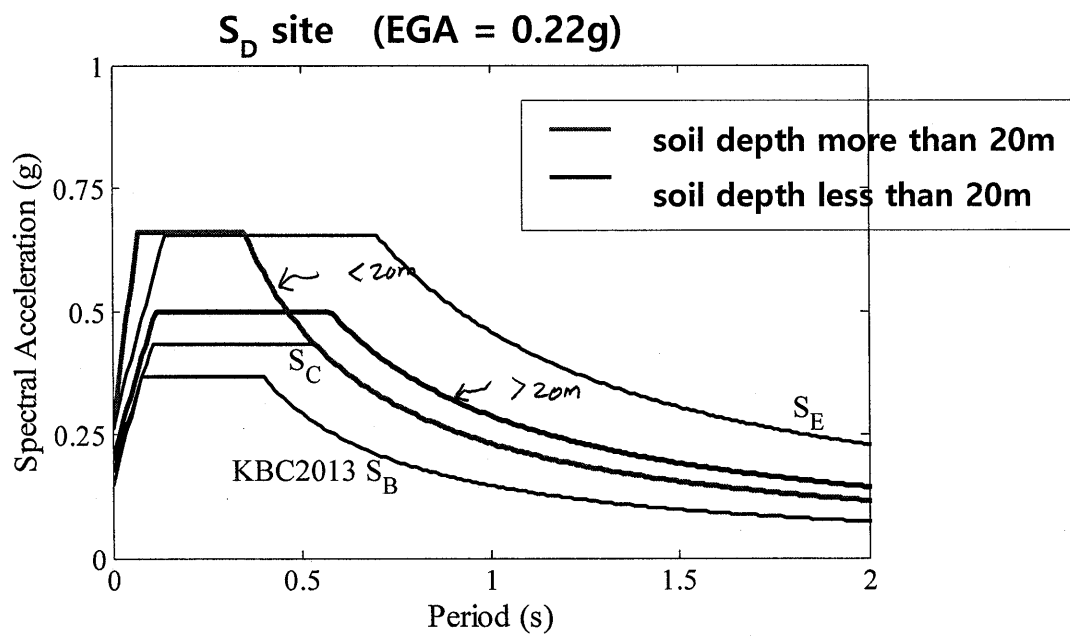
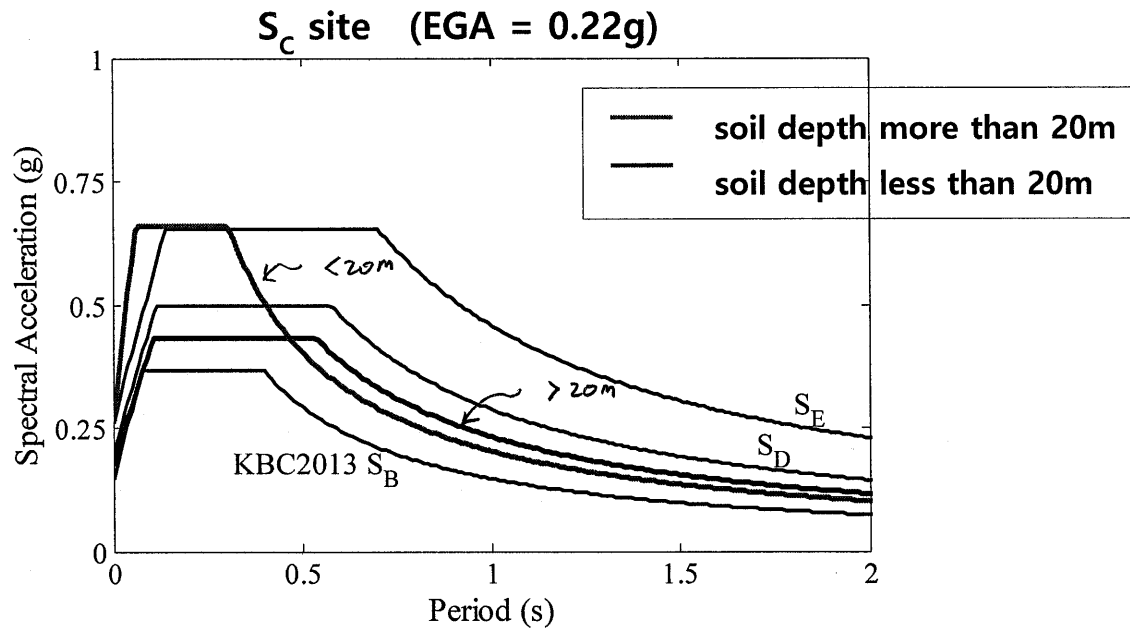
Site class		Region of seismicity		
		$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$
S_A		0.8	0.8	0.8
S_B		1.0	1.0	1.0
S_C	more than 20 m depth to rock	1.2	1.2	1.1
	less than 20 m depth to rock	1.4	1.4	1.3
S_D	more than 20 m depth to rock	1.6	1.4	1.2
	less than 20 m depth to rock	1.7	1.5	1.3
S_E		2.5	1.9	1.3

* S_s is 2.5 times the s defined for the design response spectral acceleration (Eq. 0306.3.1). For the values between the specified S_s shown above, linear interpolation shall be used.

<Table 0306.3.4> SITE COEFFICIENT FOR 1-SECOND PERIOD, F_v

Site Class		Region of Seismicity		
		$S \leq 0.1$	$S = 0.2$	$S = 0.3$
S_A		0.8	0.8	0.8
S_B		1.0	1.0	1.0
S_C	more than 20 m depth to rock	1.7	1.6	1.5
	less than 20 m depth to rock	1.5	1.4	1.3
S_D	more than 20 m depth to rock	2.4	2.0	1.8
	less than 20 m depth to rock	1.7	1.6	1.5
S_E		3.5	3.2	2.8

* S is the value used for the design response spectral acceleration (Eq. 0306.3.2). For the values between the specified S shown above, linear interpolation shall be used.



7. Importance factor

Buildings are classified according to their use and size. As the importance increases, the design earthquake load increases.

Importance category	Seismic Use Group	Importance factor (I_E)
S	S	1.5
1	1	1.2
2, 3	2	1.0

0103.1 Importance (S)

- (1) The **storage or disposal facilities** of dangerous materials with gross floor area of more than 1,000 m²
- (2) National or local government buildings, foreign embassies, **fire departments, power plants, broadcasting stations and telecommunication offices** with gross floor area more than 1,000 m²
- (3) **General hospitals, hospitals with operating rooms** or emergency rooms

0103.2 Importance (1)

- (1) The storage or disposal facilities of dangerous materials with gross floor area less than 1,000m²
- (2) National or local government buildings, foreign embassies, fire departments, power plants, broadcasting stations and telecommunication offices with gross floor area less than 1,000 m²
- (3) Concert halls, assembly halls, exhibition halls, sports facilities, sales facilities and transportation facilities with gross floor area more than 5,000 m² (Cargo terminals and delivery facilities are not included.)
- (4) Facilities for children, welfare facilities for the aged, social welfare facilities, and workers' welfare facilities
- (5) Accommodations, studio apartment buildings, dormitories and apartments with more than 5 stories

(6) Schools

(7) Hospitals without both operating rooms and emergency rooms, other medical facilities with gross floor area more than 1,000 m², which are not classified into Importance (S)

0103.3 Importance (2)

(1) Buildings which are not classified into Importance (S), (1) and (3).

0103.4 Importance (3)

(1) Agricultural facilities, and small warehouses

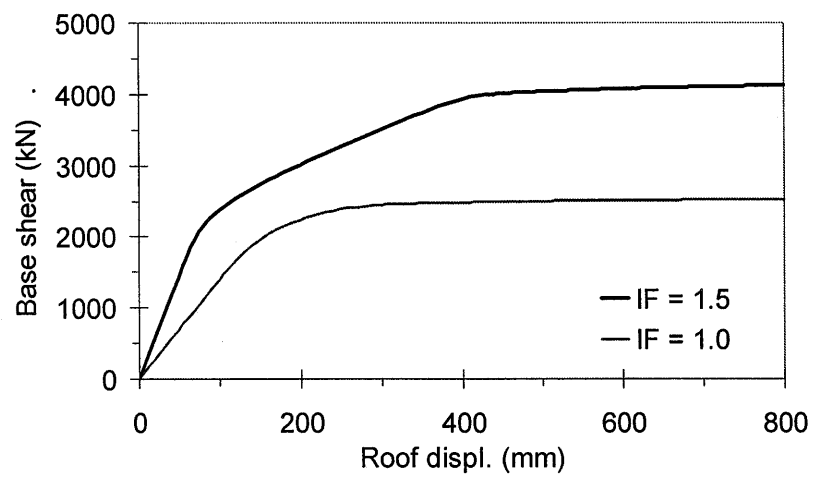
(2) Temporary structures

건물의 용도에 따라서 세 가지의 등급으로 구분되며 지진하중계산시에 해당 중요도계수를 곱하여, 중요도가 높은 건물일수록 큰 지진하중에 대하여 설계한다. 특등급에 해당되는 건물은 지진발생시 복구에 반드시 필요한 구조물 또는 해당구조물의 피해로 인하여 2 차적인 피해가 우려되는 건축물이다.

Acceptable Risk Levels

Hwang et al., 1993, "Seismic LRFD Criteria for RC MRF Buildings", J. of Str. Eng., ASCE

Building category (1)	Target Limit-State Probability	
	First yielding ($PF_{Y,T}$) (per year) (2)	Collapse ($PF_{C,T}$) (per year) (3)
Ordinary buildings	1/50	1/1,000
High-risk buildings	1/100	1/2,000
Essential buildings	1/100	1/5,000

Example building - SAC project, 3 story Seattle

8. EQ design category

As the importance of the building and peak ground acceleration increase, the design category varies from A to D. For the high category such as C and D, more accurate and sophisticate analysis methods such as dynamic analysis should be used to enhance the safety of the structure.

Value of S_{DS}	Seismic Use Group		
	S	1	2
$0.50 \leq S_{D1}$	D	D	D
$0.33 \leq S_{D1} < 0.50$	D	C	C
$0.17 \leq S_{D1} < 0.33$	C	B	B
$S_{D1} < 0.17$	A	A	A

Value of S_{D1}	Seismic Use Group		
	S	1	2
$0.20 \leq S_{D1}$	D	D	D
$0.14 \leq S_{D1} < 0.20$	D	C	C
$0.07 \leq S_{D1} < 0.14$	C	B	B
$S_{D1} < 0.07$	A	A	A

9. Equivalent Static procedure

Calculation of base shear

$$V = C_s W$$

W = total dead load + additional load listed below

- 1) 25% of floor Live load in storage and warehouse
- 2) 0.5 kN/m² (minimum) of partition load
- 3) total weight of permanent equipment

$$C_s = \frac{S_{D1}}{\left[\frac{R}{I_E} \right] T} \leq \frac{S_{DS}}{\left[\frac{R}{I_E} \right]} \geq 0.01$$

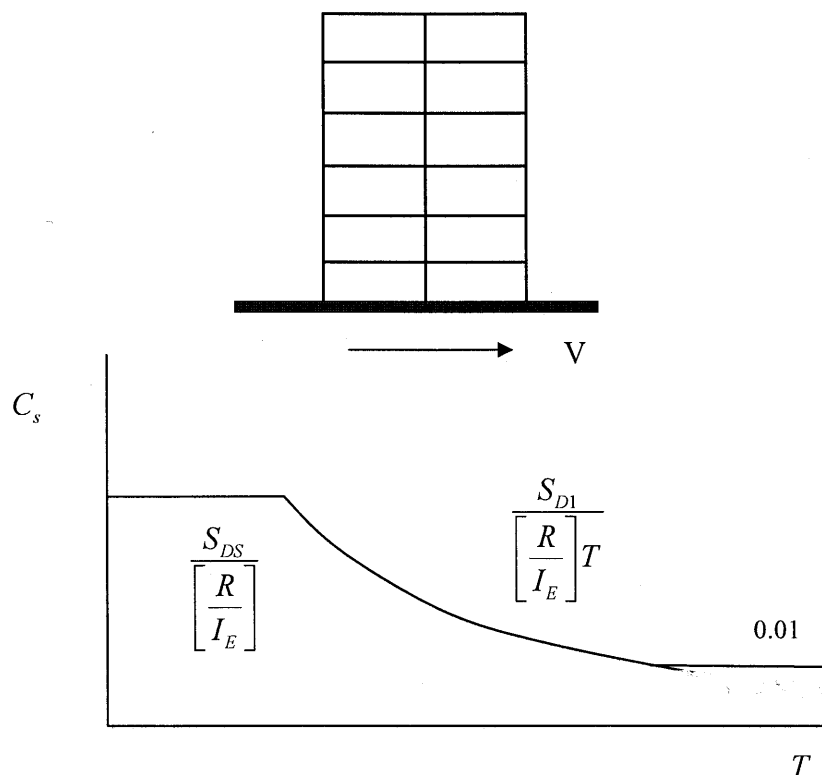
I_E = importance factor

R = response modification factor (related to ductility and over-strength)

S_{DS} = short period spectral acceleration (constant acceleration)

S_{D1} = 1 second spectral acceleration (constant velocity)

T = dynamic period of structure



Estimation of dynamic period of buildings

Empirical method

$$T_a = C_T h_n^{3/4} \quad \text{for beam-column structures}$$

$$C_T = 0.085: \text{steel moment frame}$$

$$0.073: \text{RC moment frame, eccentric braced steel frame}$$

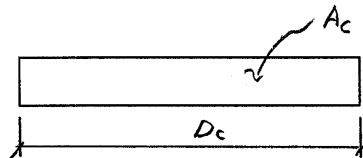
$$0.049: \text{others}$$

$$h_n = \text{height of building (m)}$$

$$T_a = 0.0743 h_n^{3/4} / \sqrt{A_c} \quad \text{for wall structures}$$

$$A_c = \sum A_c [0.2 + (D_c / h_n)^2] \quad D_c / h_n \leq 0.9$$

Wall cross-section



Rayleigh method

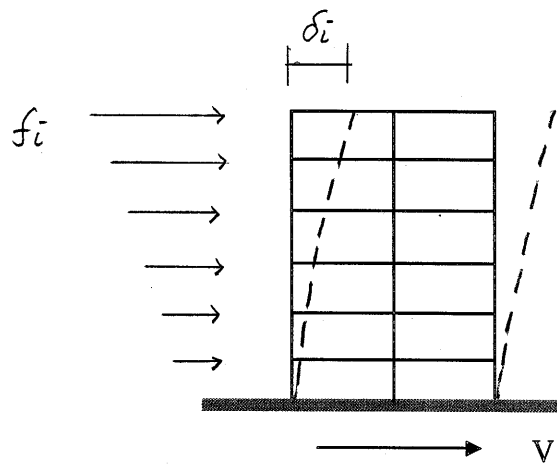
$$T = 2\pi \sqrt{\frac{\sum w_i \delta_i^2}{g \sum f_i \delta_i}}$$

f_i = applied lateral force

δ_i = corresponding displacement

$$T_i = 2\pi \sqrt{\frac{M_i^*}{K_i^*}} = 2\pi \sqrt{\frac{\underline{\phi}_i^T \underline{\mathbf{M}} \underline{\phi}_i}{\underline{\phi}_i^T \underline{\mathbf{K}} \underline{\phi}_i}} \approx 2\pi \sqrt{\frac{\sum w_i \delta_i^2}{g \sum f_i \delta_i}}$$

$$c \underline{\delta}_i \approx \underline{\phi}_i, \quad \underline{\mathbf{K}} \underline{\phi}_i \approx c f_i$$



Dynamic analysis (modal analysis)

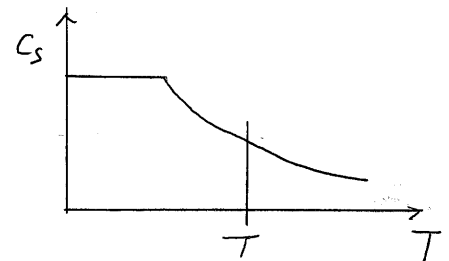
T can be obtained from modal analysis

1st eigenvalue of the structure (1st Mode)

Limitation

$$T \leq C_u T_a \quad C_u = \text{maximum period factor} = 1.4 \sim 1.7$$

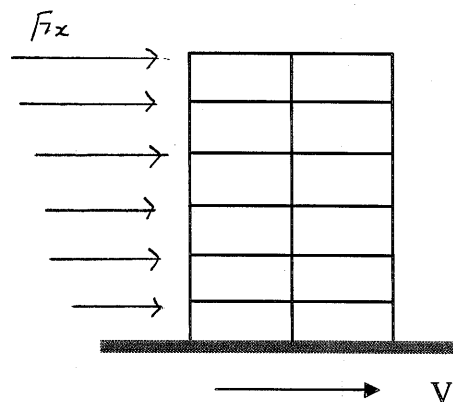
Overestimated T results in smaller earthquake load.



Story lateral load

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k}$$



lateral load distribution shape

$k=1$ for $T \leq 0.5$ → interpolation

$k=2$ for $T \geq 2.5$



$k=1$



$k=2.0$

Response modification factor

The linear elastic spectra need to be modified considering the followings

1) over-strength

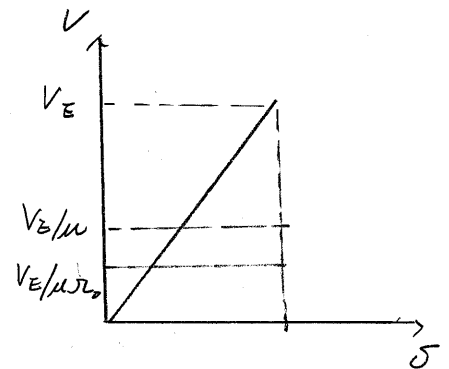
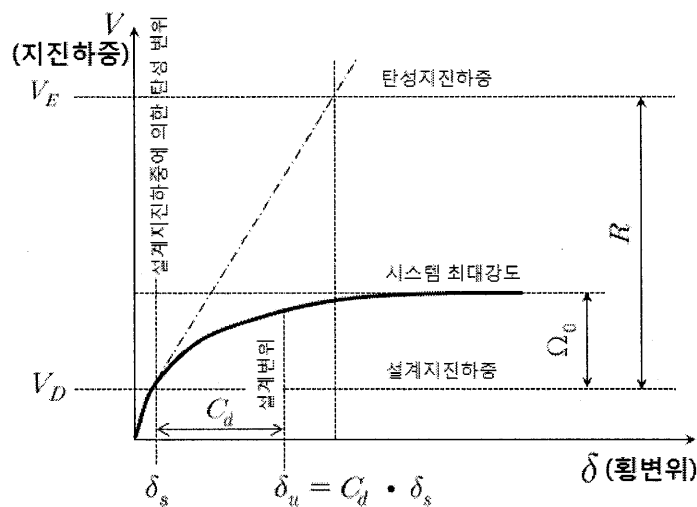
The actual strength of structures is significantly greater than the design strength for the following reasons : material overstrength, strength reduction factor, load factor (for other loads), moment(or force) redistribution (elastic analysis vs inelastic analysis), and etc.

Generally, the actual strength is greater than two times the design strength.

2) ductility

As the ductility capacity of the structure increases, the earthquake load can be decreased.

Considering the two factors, the values of the linear elastic response spectra can be decreased. The relevant factor is defined as the R factor in KBC and IBC.



기준

해설

<표 0306.6.1> 지진력저항시스템에 대한 설계계수

기본 지진력저항시스템 ¹⁾	설계계수			시스템의 제한과 높이(m) 제한		
	반응수정 계수 R	시스템 초과강도 계수 Ω_0	변위증폭 계수 C_d	내진설계 범주 A 또는 B	내진설계 범주 C	내진설계 범주 D
1. 내력벽시스템						
1-a. 철근콘크리트 특수전단벽	5	2.5	5	-	-	-
1-b. 철근콘크리트 보통전단벽	4	2.5	4	-	-	60
1-c. 철근보강 조적 전단벽	2.5	2.5	1.5	-	60	불가
1-d. 무보강 조적 전단벽	1.5	2.5	1.5	-	불가	불가
2. 건물골조시스템						
2-a. 철골 편심가새골조 (링크 타단 모멘트 저항 집합)	8	2	4	-	-	-
2-b. 철골 편심가새골조 (링크 타단 비모멘트 저항집합)	7	2	4	-	-	-
2-c. 철골 특수중심가새골조	6	2	5	-	-	-
2-d. 철골 보통중심가새골조	3.25	2	3.25	-	-	-
2-e. 합성 편심가새골조	8	2	4	-	-	-
2-f. 합성 특수중심가새골조	5	2	4.5	-	-	-
2-g. 합성 보통중심가새골조	3	2	3	-	-	-
2-h. 합성 강판전단벽	6.5	2.5	5.5	-	-	-
2-i. 합성 특수전단벽	6	2.5	5	-	-	-
2-j. 합성 보통전단벽	5	2.5	4.5	-	-	60
2-k. 철골 특수강판전단벽	7	2	6	-	-	-
2-l. 철골 좌굴방지가새골조 (모멘트 저항 집합)	8	2.5	5	-	-	-
2-m. 철골 좌굴방지가새골조 (비모멘트 저항 집합)	7	2	5.5	-	-	-
2-n. 철근콘크리트 특수전단벽	6	2.5	5	-	-	-
2-o. 철근콘크리트 보통전단벽	5	2.5	4.5	-	-	60
2-p. 철근보강 조적 전단벽	3	2.5	2	-	60	불가
2-q. 무보강 조적 전단벽	1.5	2.5	1.5	-	불가	불가
3. 모멘트-저항골조 시스템						
3-a. 철골 특수모멘트골조	8	3	5.5	-	-	-
3-b. 철골 중간모멘트골조	4.5	3	4	-	-	-
3-c. 철골 보통모멘트골조	3.5	3	3	-	-	-
3-d. 합성 특수모멘트골조	8	3	5.5	-	-	-
3-e. 합성 중간모멘트골조	5	3	4.5	-	-	-
3-f. 합성 보통모멘트골조	3	3	2.5	-	-	-
3-g. 합성 반강점모멘트골조	6	3	5.5	-	-	-

<표 0306.6.1> 지진력저항시스템에 대한 설계계수 (계속)

기본 지진력저항시스템 ¹⁾	설계계수			시스템의 제한과 높이(m) 제한		
	반응수정 계수 R	시스템 초과강도 계수 Ω_0	변위증폭 계수 C_d	내진설계 범주 A 또는 B	내진설계 범주 C	내진설계 범주 D
3-h. 철근콘크리트 특수모멘트골조	8	3	5.5	—	—	—
3-i. 철근콘크리트 중간모멘트골조	5	3	4.5	—	—	—
3-j. 철근콘크리트 보통모멘트골조	3	3	2.5	—	—	불가
4. 특수모멘트골조를 가진 이중골조시스템						
4-a. 철골 편심가새골조	8	2.5	4	—	—	—
4-b. 철골 특수중심가새골조	7	2.5	5.5	—	—	—
4-c. 합성 편심가새골조	8	2.5	4	—	—	—
4-d. 합성 특수중심가새골조	6	2.5	5	—	—	—
4-e. 합성 강판전단벽	7.5	2.5	6	—	—	—
4-f. 합성 특수전단벽	7	2.5	6	—	—	—
4-g. 합성 보통전단벽	6	2.5	5	—	—	—
4-h. 철골 좌굴방지가새골조	8	2.5	5	—	—	—
4-i. 철골 특수강판전단벽	8	2.5	6.5	—	—	—
4-j. 철근콘크리트 특수전단벽	7	2.5	5.5	—	—	—
4-k. 철근콘크리트 보통전단벽	6	2.5	5	—	—	—
5. 중간 모멘트골조를 가진 이중골조시스템						
5-a. 철골 특수중심가새골조	6	2.5	5	—	—	—
5-b. 철근콘크리트 특수전단벽	6.5	2.5	5	—	—	—
5-c. 철근콘크리트 보통전단벽	5.5	2.5	4.5	—	—	60
5-d. 합성 특수중심가새골조	5.5	2.5	4.5	—	—	—
5-e. 합성 보통중심가새골조	3.5	2.5	3	—	—	—
5-f. 합성 보통전단벽	5	3	4.5	—	—	60
5-g. 철근보강 조적 전단벽	3	3	2.5	—	60	불가
6. 역추형 시스템						
6-a. 캔틸레버 기둥 시스템	2.5	2.0	2.5	—	—	10
6-b. 철골 특수모멘트골조	2.5	2.0	2.5	—	—	—
6-c. 철골 보통모멘트골조	1.25	2.0	2.5	—	—	불가
6-d. 철근콘크리트 특수모멘트골조	2.5	2.0	1.25	—	—	—
7. 철근콘크리트 보통 전단벽-골조 상호 작용 시스템	4.5	2.25	4	—	—	60
8. 강구조기준의 일반규정만을 만족하는 철골구조시스템	3	3	3	—	—	60
9. 콘크리트기준의 일반규정만을 만족하는 철근콘크리트구조 시스템 ²⁾	3	3	3	—	—	30

1) 시스템별 상세는 각 재료별 설계기준 및 또는 신뢰성 있는 연구기관에서 실시한 실험, 해석 등의 입증자료를 따른다.

2) 철근콘크리트설계기준의 일반규정이란 5장에서 0520절을 제외한 나머지 규정을 의미한다.

<Table 0306.6.1> DESIGN COEFFICIENTS FOR SEISMIC-FORCE-RESISTING SYSTEMS

Seismic-force-resisting system ¹⁾	Design coefficients			Limitations of building height (m)		
	Response modification coefficient R	System over strength factor Ω_0	Displacement amplification factor C_d	Seismic Design Category A or B	Seismic Design Category C	Seismic Design Category D
1. Bearing wall systems						
1-a. Special reinforced concrete shear walls	5	2.5	5	-	-	-
1-b. Ordinary reinforced concrete shear walls	4	2.5	4	-	-	60
1-c. Reinforced masonry shear walls	2.5	2.5	1.5	-	60	NP
1-d. Unreinforced masonry shear walls	1.5	2.5	1.5	-	NP	NP
2. Building frame systems						
2-a. Steel eccentrically braced frames (moment resisting column-link connections)	8	2	4	-	-	-
2-b. Steel eccentrically braced frames (non-moment resisting column-link connections)	7	2	4	-	-	-
2-c. Special steel concentrically braced frame	6	2	5	-	-	-
2-d. Ordinary steel concentrically braced frame	3.25	2	3.25	-	-	-
2-e. Composite eccentrically braced frames.	8	2	4	-	-	-
2-f. Special composite concentrically braced frames	5	2	4.5	-	-	-
2-g. Ordinary composite concentrically braced frames	3	2	3	-	-	-
2-h. Composite steel plate shear walls	6.5	2.5	5.5	-	-	-
2-i. Special composite shear walls	6	2.5	5	-	-	-
2-j. Ordinary composite shear walls	5	2.5	4.5	-	-	60
2-k. Special steel plate shear walls	7	2	6	-	-	-
2-l. Buckling-restrained braced frames (moment-resisting beam-column connections)	8	2.5	5	-	-	-
2-m. Buckling-restrained braced frames (non-moment-resisting beam-column connections)	7	2	5.5	-	-	-
2-n. Special reinforced concrete shear	6	2.5	5	-	-	-

walls						
2-o. Ordinary reinforced concrete shear walls	5	2.5	4.5	-	-	60
2-p. Reinforced masonry shear walls ²⁾	3	2.5	2	-	60	NP
2-q. Unreinforced masonry shear walls	1.5	2.5	1.5	-	NP	NP
3. Moment-resisting frame systems						
3-a. Special steel moment frames	8	3	5.5	-	-	-
3-b. Intermediate steel moment frames	4.5	3	4	-	-	-
3-c. Ordinary steel moment frames	3.5	3	3	-	-	-
3-d. Special composite moment frames	8	3	5.5	-	-	-
3-e. Intermediate composite moment frames	5	3	4.5	-	-	-
3-f. Ordinary composite moment frames	3	3	2.5	-	-	-
3-g. Composite partially restrained moment frames	6	3	5.5	-	-	-
3-h. Special reinforced concrete moment frames	8	3	5.5	-	-	-
3-i. Intermediate reinforced concrete moment frames	5	3	4.5	-	-	NP
3-j. Ordinary reinforced concrete moment frames	3	3	2.5	-	-	NP
4. Dual systems with special moment frames						
4-a. Steel eccentrically braced frames	8	2.5	4	-	-	-
4-b. Special steel concentrically braced frames	7	2.5	5.5	-	-	-
4-c. Composite eccentrically braced frames	8	2.5	4	-	-	-
4-d. Special composite concentrically braced frames	6	2.5	5	-	-	-
4-e. Composite steel plate shear walls	7.5	2.5	6	-	-	-
4-f. Special composite shear walls with steel elements	7	2.5	6	-	-	-
4-g. Ordinary composite shear walls	6	2.5	5	-	-	-
4-h. Buckling-restrained braced frames	8	2.5	5	-	-	-
4-i. Special steel plate shear walls	8	2.5	6.5	-	-	-
4-j. Special reinforced concrete shear walls	7	2.5	5.5	-	-	-
4-k. Ordinary reinforced concrete shear walls	6	2.5	5	-	-	-
5. Dual systems with intermediate moment frames						

5-a. Special steel concentrically braced frames	6	2.5	5	-	-	-
5-b. Special reinforced concrete shear walls	6.5	2.5	5	-	-	60
5-c. Ordinary reinforced concrete shear walls	5.5	2.5	4.5	-	-	-
5-d. Special composite concentrically braced frames	5.5	2.5	4.5	-	-	-
5-e. Ordinary composite concentrically braced frames	3.5	2.5	3	-	-	60
5-f. Ordinary composite shear walls with steel elements	5	3	4.5	-	-	NP
5-g. Reinforced masonry shear walls	3	3	2.5	-	60	NP
6. Inverted pendulum systems						
6-a. Cantilever column systems	2.5	2	2.5	-	-	-
6-b. Special steel moment frames	2.5	2	2.5	-	-	NP
6-c. Ordinary steel moment frames	1.25	2	2.5	-	-	
6-d. Special reinforced concrete moment frames	2.5	2	1.25			60
7. Shear wall-frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls	4.5	2.25	4	-	-	60
8. Steel systems not specifically detailed for seismic resistance	3	3	3	-		30
9. Reinforced concrete systems not specifically detailed for seismic resistance ²⁾	3	3	3	-		30

1) Details on each system are determined by the corresponding design code or analytical and experimental results performed by authorized research facilities.

2) The reinforced concrete systems are designed according to the design requirements contained in Chapter 5 except for Section 0520.

3) NP: Not Permitted

Definitions of structural system

Bearing wall system

Wall – slab system. Popular system for apartment buildings in Korea

Both gravity load and lateral load is resisted by bearing walls.

Moment resisting frame

Beam and column system.

Lateral load is resisted by moment resisting actions of beams and columns

Building system

Primary lateral load resisting system + moment frame

Primary lateral load resisting system = walls and braces

All lateral load is resisted by walls or braces

Gravity load is resisted by moment frame

Dual system

Superficially, this system is similar to the Building system.

Primary lateral load resisting system + moment frame

A part of lateral load can be resisted by moment frame

The moment frame should resist at least 25% of the lateral load.

Structural elements

Concrete walls : rectangular wall,

barbell-shaped wall : wall with two end columns

core wall : generally located at the center or ends of floor plan
in a building

Steel braces ;

Concentric braces : greater strength, less ductility (due to buckling)

Eccentric braces : less strength, greater ductility

Classification of structural system according to the ductility details

Special system : special ductility details are used to significantly enhance the ductility and energy dissipation capacity of the structure

Classification of structural system according to the ductility details

Special system : special ductility details are used to significantly enhance the ductility and energy dissipation capacity of the structure

Intermediate system : moderate ductility details

Ordinary system : seismic details are not used.

Torsional moment

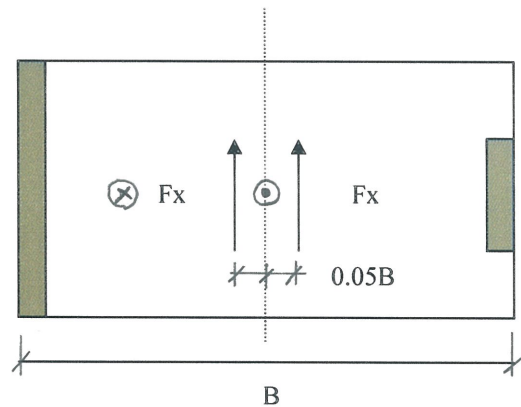
The eccentricity between the rigidity center and load center (mass center) cause torsional moment in addition to the lateral load, which cause severe damage to the structure.

Considering the uncertainty of the eccentricity, additional 5% eccentricity is added.

⊗ Rigidity center

⊙ Mass center

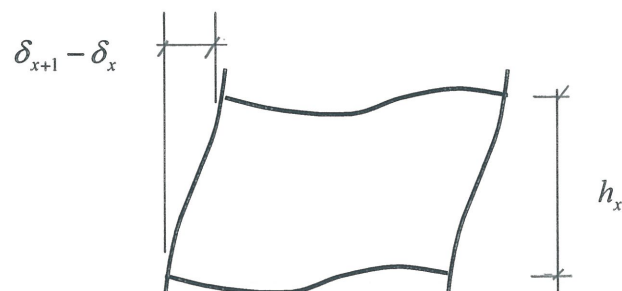
Additional (accidental eccentricity) $= 0.05B$



Inter-Story drift

$$\delta_{x+1} - \delta_x \leq \Delta_a$$

Δ_a = permissible inter-story drift



$\Delta_a / h_x = 0.01$ Importance class critical (S)

0.015 class I (1)

0.02 class II (2)

$$\delta_x = \frac{C_d \delta_{xe}}{I_e}$$

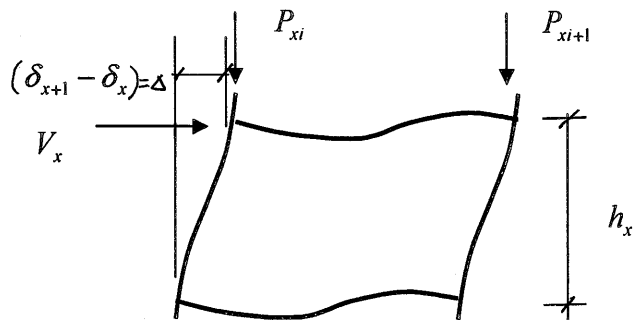
C_d = displacement factor (to consider the inelastic deformation)

δ_{xe} = lateral deflection resulting from elastic analysis

$P-\Delta$ effect

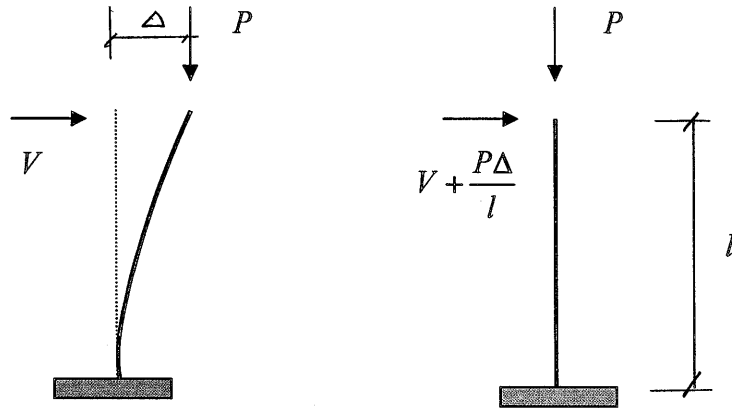
$$\delta_x = a_d \delta_x \quad a_d = \frac{1}{(1-\theta)}$$

$$\theta = \frac{\sum P_{xi} \Delta}{V_x h_{sx} C_d} \quad \frac{\Delta}{C_d} = \text{elastic deformation (1st order analysis)}$$



If $\theta \leq 0.1$, $P-\Delta$ effect can be neglected.

$$\theta \text{ must be less than } \theta_{\max} = \frac{0.5}{\beta C_d} \leq 0.25, \quad \text{generally } \beta = 1$$



$$\Delta_o = \frac{Vl^3}{3EI}, \quad \Delta = \frac{Vl^3}{3EI} \left(1 + \frac{P\Delta}{Vl} \right)$$

$$\left(1 - \frac{P\Delta_o}{Vl} \right) \Delta = \Delta_o \quad \Delta = \Delta_o \left(1 - \frac{P\Delta_o}{Vl} \right)^{-1}$$