Introduction to Earthquake Engineering and Seismic Codes in the World

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This lecture note was written by Dr. Yuji Ishiyama. He retired from Hokkaido University in 2005, and published the book "Seismic Codes and Structural Dynamics (in Japanese)"[1] in 2008. This is the English version of the book, but only included are Part I and Part IV. (Part II and Part III are in a separate lecture note that include the fundamentals of structural dynamics.)

If you have questions, suggestions, or comments on this lecture note, please e-mail to the author. Thank you in advance for taking the time and interest to do so.

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Chapter 1

Introduction to Earthquake Engineering

1.1 Earthquake Damage to Buildings

1) Damage to First Story

Typical earthquake damage to buildings is the collapse of the first story (Fig.1.1, Fig.1.2). The top of a building vibrates most severely when it it subjected to earthquake ground motions, and the seismic force at the top becomes the largest. But the seismic force at each story is transmitted from the top to the base, and eventually to the ground. Consequently, lower stories must sustain the sum of seismic forces of upper stories. The sum of seismic forces from the top to the story concerned is called the "seismic shear force". Since the seismic shear force becomes the largest at the lowest story, the collapse of the first story is typical earthquake damage.

Besides the shear force becomes the largest at the first story, it is usually difficult to install shear walls, which are very effective against earthquakes, at the first story. This is because the first story is frequently used for garages or shops and larger space and openings are required. Therefore, the earthquake resistant capacity at the first story tends to be smaller than upper stories, and the first story collapse is very common during any earthquakes.

Figure 1.1: Typical earthquake damage: collapse of the soft and weak first story
(1978 Miyagi-ken-oki Earthquake, two 3 story RC buildings)

Figure 1.2: Collapse of the (soft and weak) first story
(1995 Hyogo-ken-nanbu Earthquake)

2) Damage to Uppermost Story

The top of a building vibrates most severely during earthquakes, and then the seismic force at the top becomes the largest. Therefore, the overturning of furniture concentrates to the upper stories (the left of Fig.1.3). Becase of the same reason the collapse of penthouses, appendages and water tanks at the top of buildings is also frequently observed earthquake damage (the right of Fig.1.3).

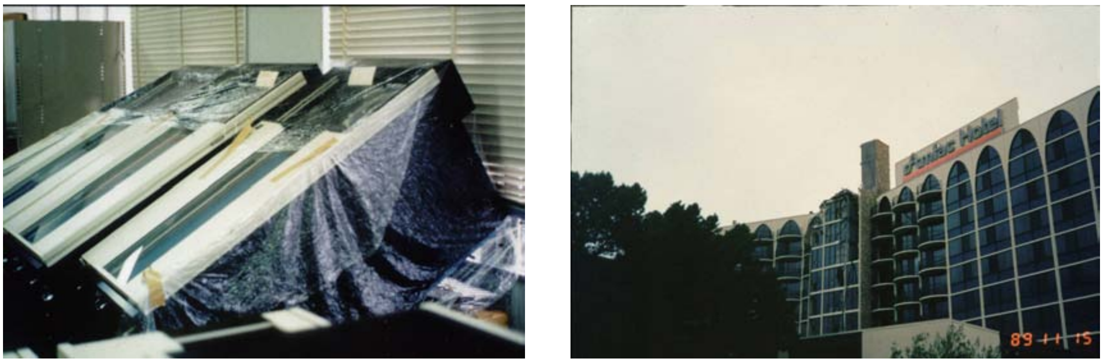


Figure 1.3: Overturning of computers and collapse of water tank
(1978 Miyagi-ken-oki Earthquake, and 1989 Loma Prieta, California Earthquake)

3) Damage to Mid-Story

During the 1995 Hyogo-ken-nanbu (Kobe) Earthquake, several buildings suffered from mid-story collapse (Fig.1.4), which is rather rare earthquake damage. The reason of the damage is considered that those buildings were designed using an old seismic code, in which the distribution of seismic forces had not been properly considered, and also there was structural discontinuity along the heigh of the buildings, i.e. the steel frame reinforced concrete for lower stories and reinforced concrete for upper stories.

Figure 1.4: Mid-story collapse
(1995 Hyogo-ken-nanbu Earthquake)

4) Damage to Wooden Buildings

Most houses in Japan are made of wood frame construction with shear walls where braces are installed. The earthquake resistant capacity of those houses depends mostly on the amount of the shear walls. But, it is difficult to intall enough shear walls into two (X and Y) directions especially for a narrow site in urban area. And this is one of the reasons to increase earthquake damage to those houses (the left of Fig.1.5).

Since narrow wooden boards nailed to wood frame can not resist horizontal forces (the right of Fig.1.5), braces, plywood and/or sheathing boards nailed to the frame are required. The size of braces is important, but the joint detail of braces is also important (the left of Fig.1.6). In case nailing may not be enough, it is recommended to use metal fasteners at joints. Wood panels of plywood and/or sheathing boards nailed to the frame along the fringes of the panels are very effective to resist earthquake forces (the right of Fig.1.6).

Figure 1.5: First story collapse of a wooden house and detail of damaged wall sheathing
(1995 Hyogo-ken-nanbu Earthquake)

Figure 1.6: Damage to brace joints and strengthening using plywood
(1978 Miyagi-ken-oki Earthquake, and 1989 Loma Prieta, California Earthquake)

5) Damage to Non-Structural Elements

In many earthquakes, damage to structural elements may be minor, but roof tiles, exterior finishings, concrete block fences, etc. are extensively damaged even by moderate earthquakes (Fig.1.7, Fig.1.8). Overturning of furniture may injure people, hinder evacuation, cause fires, etc. The damage to non-structural elements may occur even during minor earthquakes, and can not be ignored from an economical point of view.

Figure 1.7: Fallen concrete curtain wall panels and jammed doors
(1978 Miyagi-ken-oki Earthquake)

6) Damage Caused by Soil Failure

It has been well known that the earthquake damage to buildings bocomes severer in case they stand on soft soil. When land slide and soil failure occur, buildings may collapse (Fig.1.9). It is impossible to protect buildings only by increasing the earthquake resistant capacity of those buildings. Therefore, it is essential to consider the earthquake resistant capacity and stability of the site, selecting a construction site and reclaiming land for constructing buildings.

Figure 1.8: Overturning of concrete block fence and furniture
(1978 Miyagi-ken-oki Earthquake)

Figure 1.9: Damage to buildings caused by soil failure
(1974 Izu-ohshima-kinkai Earthquake, and 1993 Kushiro-oki Earthquake)

7) Damage to Non-Engineered Buildings

Most buildings in the world are constructed wituout any intervention of engineers. Some examples of so-called non-engineered buildings are of adobe (sun-burned mud block construction), tapial (cast-in-place mud construction, the left of Fig.1.10), unreinforced brick masonry (the right of Fig.1.10), unreinforced stone masonry, etc.

Even in Japan, many old wooden houses may be classified as non-engineered buildings, which do not satisfy current seismic regulations, e.g. necessary amount of shear walls.

Many engineers and researchers have much interests to large-scale structures, high-rise buildings, high-tech structures, etc., but only few engineers or researchers pay attention to the non-engineered buildings. Without reducing the damage to non-engineered buildings, it is impossible to reduce the earthquake damage in the world. And younger engineers and researchers are expected to work on this problem.

Figure 1.10: Damage to non-engineered buildings
(1990 Rioja, Peru Earthquake, and 2006 Mid Jawa, Indonesia Earthquake)

1.2 Behavior of Buildings during Earthquakes

1) Vibration of Buildings and Seismic Forces

Earthquake ground motions are complicated, but the motion of buildings are relatively simple. Fig.1.11 shows acceleration time histories of the 30 seconds recorded in the 1978 Miyagiken-oki Earthquake at a 9 story reinforced concrete building of Tohoku University, Sendai, Japan. The upper graph is the record of the 9th story and the lower is of the 1st story. Both are shown in the same scale and the unit of Y axis is "gal" or "cm/s^2". It can be seen that the amplitude of upper graph is larger than that of the lower one. And the maximum acceleration of the upper graph is 1040 gal, that is four times larger than that of the lower graph of 260 gal. Emperically it is said that the amplification ratio between the top and the base of a building is approximately three, although the ratio is four in Fig.1.11.

The time history of the lower graph of Fig.1.11 is rather complicated, but the upper graph shows that peaks appear periodically. The duration between two adjacent peaks is called the natural period of the bulding, and the upper graph of Fig.1.11 indicates that the natural period of this building is one second.

When the building is subjected to earthquake ground motions, the motion of the building is similar to harmonic with gradual change of amplitude. The motion of the building changes for different earthquakes, but it always vibrates with its natural period. This is the reason why it is called the "natural " period.

The natural period of a building is approximately proportional to the number of stories and "0.1 times the total number of stories" gives the natural period T in second as follows.†

$$T = 0.1\,n \quad \text{(s)} \tag{1.1}$$

where, n is the number of stories of the building above the ground level.

Using the above formula, the natural period of a two story building is 0.2 (s), 1.0 (s) for 10 story building, and 5.0 (s) for a 50 story high-rise building. Since the natural period also

†ASCE 7 of U.S. includes this formula (see Eq.(3.73) on p.100) for designing buildings.

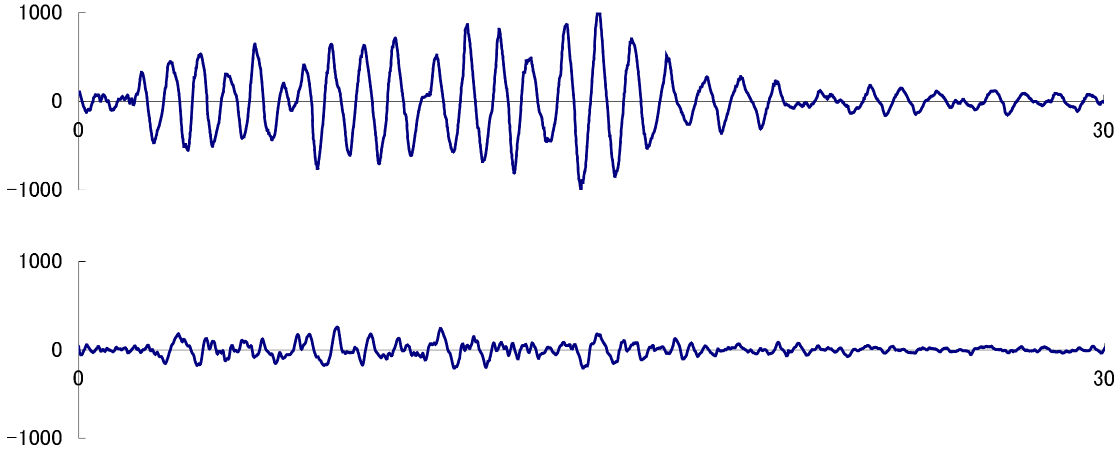


Figure 1.11: Recorded earthquake motions in a 9 story reinforced concrete building
(1978 Miyagi-ken-oki earthquake, acceleration in gal (cm/s^2) during 30 seconds)

depends on the structural types, etc., Japanese code (Building Standard Law of Japan) uses another formula to give natural period T as follows.

$$T = (0.02 + 0.01\lambda)\,h \quad \text{(s)} \tag{1.2}$$

where, h is the height (m) of the building, and λ is the ratio of the steel or wooden part height to the total height of the building. Therefore the above formula becomes as follows for reinforced concrete buildings and for steel buildings or wooden buildings.

$$\begin{aligned} T &= 0.02\,h \;:\; \text{for reinforced concrets buildings} \\ T &= 0.03\,h \;:\; \text{for steel and wooden buildings} \end{aligned} \tag{1.3}$$

The above formulea are empirically obtained, and the natural period of a 10 m high building becomes 0.2 (s) for reinforced concrete buildings and 0.3 (s) for steel or wooden buildings.

The seismic force is the inertia forces, that acts every part of the building, caused by earthquake ground accelerations. The seismic force P is obtained as the product of the mass m and the acceleration a or the product of the seismic coefficient k and the weight w as follows.

$$P = m\,a \tag{1.4}$$

$$P = k\,w \tag{1.5}$$

2) Seismic Force and Seismic Shear Force

Earthquake ground motions exert seismic forces to buildings not only horizontally but also vertically. Since the buildings usually have enough capacity against vertical motions, most seismic codes stipulate only horizontal seismic forces.

If the building moves as a rigid body along with the ground motion, seismic forces and seismic coefficients become uniform from the top to the base as shown in Fig.1.12 a. The seismic forces of the upper stories are transmitted to lower stories and finally to the ground

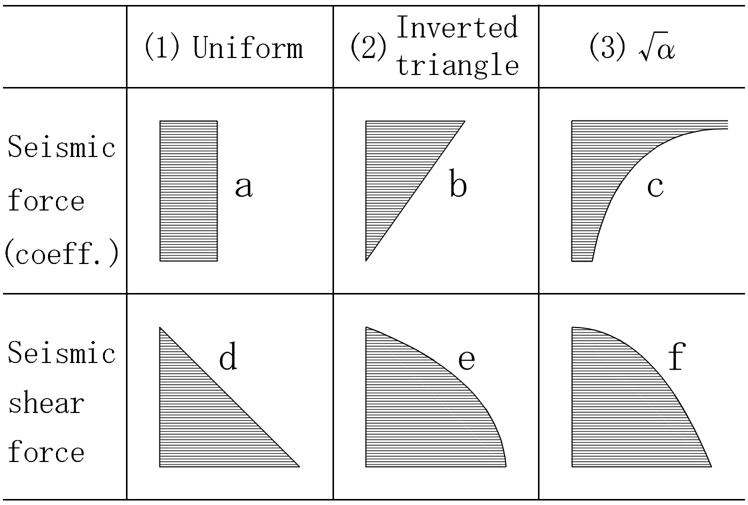


Figure 1.12: Distributions of seismic forces (coefficients) and seismic shear forces

through the foundation, and the seismic shear forces are the sum of seismic forces of upper stories. In case the seismic forces are uniformly distributed, the seismic shear forces increase linearly from top to base (Fig.1.12 d). This distribution (Fig.1.12 a, d) is called (1) uniform distribution of seismic forces.

Since the building is not rigid, the upper stories are subjected to larger seismic forces. If the response displacements increase linearly from the base to the top, so as the seismic forces (Fig.1.12 b). Then the distribution of seismic shear forces becomes as a palabola where the vertix locates at the bottom or which opens to the left (Fig.1.12 e). This distribution (Fig.1.12 b, e) is called (2) inverted triangle distribution of seismic forces.

3) Vibration of Buildings and Eigenmodes of Vibration

When the building vibrates, the motion becomes the combination of various modes of vibration (eigenmodes, Fig.1.13). Each mode of vibration has its own natural period and mode shape. The longest natural period is called the fundamental (first) narural period, the second longest is the second natural period, etc. And the corresponding mode shapes are called the funamental (first) mode, the second mode, etc. When the building is subjected to the earthquake ground motions, it vibrates with the combination of all modes and each mode changes its amplitude but its natural period never changes, while the response of the building remains within the elastic range. Generally the fundamental mode is the most influential as shown in the upper graph of Fig.1.11, where the motion looks like sinusoidal vibtration of 1 (s) period gradually changing its amplitude. The 1 (s) period is the fundamental natural period of the building.

For highrise buildings, the higher mode effect becomes larger, and the top of the buildings moves as the tip of a whip. This whipping phenomenon causes overturning of furniture at upper stories, damage to penthouses and water tanks on the roof floor (Fig.1.3). The seismic forces of upperstories become very large, and the distribution of the seismic forces can be shown as Fig.1.12 c. Then the seismic shear forces distribute as a parabola where the vertex is at the top or which opens downwards (Fig.1.12 f). Defining the normalized weight α, cf. Eq.(1.7), as the

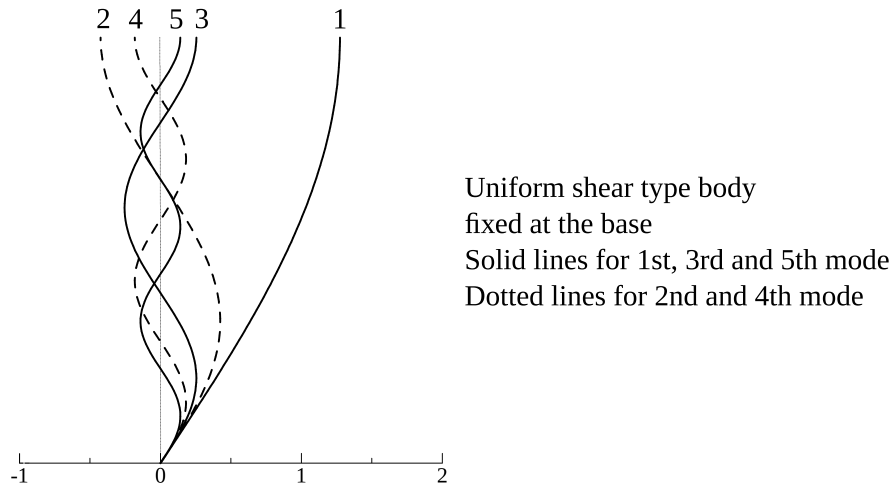


Figure 1.13: Eigenmodes (participation functions) of a unform shear type body fixed at the base

weight above the level concerned divided by the total weight above the base, the distribution of the seismic shear forces is proportional to $\sqrt{\alpha}$. Therefore this distribution can be called (3) "$\sqrt{\alpha}$ distribution" (Fig.1.12f). Incidentally the distribution of seismic forces is proportional to $1/\sqrt{\alpha}$ (Fig.1.12 c).

All three distributions shown in Fig.1.12 indicate the seismic shear forces become maximum at the base, so that the damage to the first story is very common (Fig.1.1, Fig.1.2). Furthermore the first story is frequently used as garages or shops where seismic capacity are apt to be smaller than other stories, and the damage to the first story becomes severer, sometimes causing complete collapse.

The seismic shear force at the level concerned divided by the weight above the level is called "seismic shear coefficient". In the seismic code of Japan, the distribution of the seismic shear coefficients is given by so-called A_i distribution[†] (see Fig.2.14 on p.37) as follows.

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right)\frac{2T}{1+3T} \tag{1.6}$$

where, T (s) is the fundamental natural period of the building, and α_i is the normalized weight of the level i that is defined as follows.

$$\alpha_i = \frac{\sum_{j=i}^{n} w_j}{\sum_{j=1}^{n} w_j} \tag{1.7}$$

where, n is the number of stories above the base, i and j indicate the story number, and w_j is the weight of the j-th story.

A_i distribution includes three distributions indicated in Fig.1.12, i.e. (1) unform distribution of seismic forces, (2) inverted triangular distribution of seismic forces, and (3) $\sqrt{\alpha}$ distribution. A_i distribution converges to the distribution (1) when the natural period becomes shorter for low-rise buildings, and the effects of the distributions (2) and (3) becomes larger for high-rise buildings.

[†] A_i distibution was proposed by the author some thirty years ago.

Normalized weight
From the definition of Eq.(1.7), the normalized weight α_i becomes unity (1.0) at the first story and converges to zero at upper stories, but never becomes zero. This normalized weight is very convenient to express the distributions of seismic forces and shear forces, although only Japanese code adopts the normalized weight as a seismic force parameter. Many codes in the world use height h as a parameter, but it is difficult to express (3) $\sqrt{\alpha}$ distribution with the parameter h.

1.3 History of Earthquake Resistant Technology in Japan

1) Introduction

Many big earthquakes have occurred in and around Japan since ancient times and have caused severe damage, but the scientific research on seismology or earthquake engineering began only after the Meiji Restoration in 1868. The Yokohama Earthquake occurred in 1880, and the Seismological Society of Japan was established. Since then, every big earthquake had forced to make some development in earthquake resistant technology and seismic codes. For example, the 1891 Nobi Earthquake led to the formation of the Earthquake Investigation Committee, the 1923 Great Kanto Earthquake brought about the introduction of the seismic coefficient, the 1968 Tokachi-oki Earthquake prompted the revision of shear reinforcement of reinforced concrete columns, the 1978 Miyagi-ken-oki Earthquake led to the adoption of a new seismic design method, the 1995 Hyogo-ken-nanbu (Kobe) Earthquake prompted the revision of shape factor in Building Standard Law and introduction of Seismic Retrofitting Promotion Law, etc. This section reports a brief history of earthquake resistant technology and related matters in Japan.

2) After the Meiji Restoration in 1868

(1) Yokohama Earthquake and Seismological Society of Japan

After the Meiji Restoration in 1868, the government of Japan abandoned the 230 year isolation from foreign countries and resumed overseas trade. The government invited many foreign researchers, scientists and engineers to Japan to develop Japanese industries as well as to absorb Western culture as soon as possible. The Yokohama Earthquake (M=5.5) occurred in 1880 and caused some damage. The earthquake was not very severe, but was strong enough to frighten resident researchers from overseas countries. In the same year, the Seismological Society of Japan was founded by the efforts of Dr. J. Milne (Fig.1.14), Dr. J. A. Ewing, etc., and scientific research on seismology and earthquake engineering began.

(2) Nobi Earthquake and Earthquake Investigation Committee

In 1891 the Nobi Earthquake (M=7.9, 7,273 deaths, more than 140,000 buildings† of total collapse, more than 80,000 of partial collapse) occurred and caused very severe damage. In the following year (1892), the Earthquake Investigation Committee was founded and research on

†Including houses.

Figure 1.14: Biography of J. Miln and his tomb in Hakodate, Japan
(“JOHN MILNE: FATHER OF MODERN SEISMOLOGY”
by A.L. Herbert-Gustar & P.A. Nott was published in 1980.)

seismology was accelerated. The Earthquake Investigation Committee was rechartered to the Earthquake Research Institute in 1925 which now belongs to the University of Tokyo.

The Earthquake Investigation Committee made several proposals to improve earthquake resistant capacity of wooden houses, e.g. strengthening of foundations, lessening cut for wooden members, using metal fasteners for joints, installing braces, etc. The committee formulates current concepts for improving earthquake resistant capacity of wooden houses.

(3) Buildings in Meiji Era

In Japan, most buildings have been made of wood since ancient times and technology to construct wooden buildings were traditionally succeeded by leaders of carpenters. Because of many wooden houses, whenever a fire happened, it lasted for several days and destroyed a major part of a town or a city. Therefore it was proverbially said that “Fire is a flower in Edo (old Tokyo).” Therefore, what the Meiji government demanded first was fire prevention to buildings, and not their earthquake resistant capacity. The policy such as “Ginza brick town plan” was to construct buildings in urban area made of mud, brick and masonry.

Many big earthquakes have occurred in and around Japan since ancient times, but probably because collapse of houses by earthquakes is less frequent than by fires, countermeasures for earthquakes had not been taken in Japan.

The brick and masonry construction was introduced in Japan at the end of Tokugawa Era and many buildings were constructed according to European or American structural specifications. There were almost no measures against earthquakes for those buildings. Therefore, many buidings so colled “Meiji buildings of red bricks” suffered severe damage in the Nobi Earthquake. Since then, brick buildings were reinforced with steel frame in the latter period of Meiji. Because brick construction suffered devastating damage again during the Great Kanto Earthquake, it was gradually not used for buildings.

Reinforced concrete and steel frame reinforced concrete construction started at the begining of the 20th century. Steel had been imported from foreign countries, but it was gradually produced in Japan since the beginning of the 20th century. The steel was first used in the fields of civil engineering and shipbuilding, then it came to be also used for buildings. Cement was produced in Japan since the beginning of Meiji Era. It was used in civil engineering field and then came to be used for buildings with the increase of cement production.

The San Francisco Earthquake (M=8.3, 600 deaths) occurred in the U.S. in 1906. The first World Conference on Earthquake Engineering (WCEE) was held at San Francisco in 1956, to commemorate the 50th year anniversary of the 1906 Earthquake. Since then, WCEE is held approximately every four years. Dr. Tatsutaro Nakamura and Dr. Toshikata Sano went to San Francisco to investigate the 1906 Earthquake. They came back with the conviction that reinforced concrete structures should have not only excellent fireproof capacity but also excellent earthquake resistant capacity.

3) After the 1923 Great Kanto Earthquake

(1) Great Kanto Earthquake and Seismic Coefficient

The Great Kanto Earthquake (M=7.9, more than 105,000 deaths or missing, 254,000 buildings of total or partial collapse, 447,000 buildings were burnt down) in 1923 killed more than one hundred thousand people. In the following year of 1924, the provision that "the horizontal seismic coefficient shall be at least 0.1" was added to the Urban Building Law. This quick response to revise the regulation was due to the fact that Dr. Toshikata Sano published the paper "Theory of Earthquake Resistant Buildings" in 1914, and proposed the concept of seismic coefficient based on the paper in the Earthquake Investigation Committee Report in 1916. The reason why the seismic coefficient was specified to be 0.1 is that the seismic coefficient of Tokyo was estimated to be 0.3 of the acceleration due to gravity during the Great Kanto Earthquake, and the safety factor of material strength to allowable stress was assumed to be 3.

In structural regulations of the Urban Buiding Law in 1920, strength calculation was demanded for steel and reinforced concrete buildings, but it was not required for wooden buildings. There were no regulations for wind pressure nor for seismic forces. There was, however, regulations to install braces and horizontal corner braces in three-story wooden buildings, which shows that some consideration for seismic forces was included in the Urban Building Law.

The revision of Urban Building Law in 1924 was not only to supplement the seismic coefficient, but also to increase the size of wooden columns, to install braces and knee braces for wooden buildings, etc. For masonry buildings, the revision was to install ring beams and to strengthen walls. For steel buildings, the revision was to install braces, knee braces and walls, to connect partition walls with surrounding frames, and to prohibit using hollow tiles for partition walls. For reinforced concrete buildings, the revision was to specify the length of lap joints of reinforcing bars, to arrange reinforcing bars not only in tension side but also in compression side, to increase the size of columns, and to specify minimum reinforcement ratio for columns. The revision was to increase strength and stiffness against lateral forces, and to improve joints between structural members. As for masonry and brick buildings, the height limitation became so stringent that they were hardly built after the revision.

Table 1.1: Earthquake resistant technology and related events in Japan

Year	Major Earthquakes	Related Events
1868	(Meiji Era)	Meiji Restoration
1880	Yokohama Earthquake	Seismological Society of Japan
1891	Nobi Earthquake	
1892		Earthquake Investigation Committee
1906	San Francisco Earthquake, U.S.	
1912	(Taisho Era)	
1916		Concept of seismic coefficient by Dr. Sano
1923	Great Kanto Earthquake	
1924		Introduction of the seismic coefficient ($k \geqq 0.1$)
1925		Earthquake Research Institute
1926	(Showa Era)	
1933	Sanriku-oki Earthquake	D-value method by Dr. Muto
1940	Imperial Valley Earthquake, U.S.	
	(El Centro Ground Motions)	
1945		End of World War II
1950		Building Standard Law (Seismic coefficient $k \geqq 0.2$)
1956		First World Conference on Earthquake Engineering
1963		Abolishment of height limitation (31m)
1964	Niigata Earthquake	
1968	Tokachi-oki Earthquake	High-rise Kasumigaseki Building
1971		Strengthening of shear reinforcement for RC columns
1977		New Seismic Design Method (proposed)
		Seismic Diagnosis for RC buildings
1978	Miyagi-ken-oki Earthquake	
1981		New Seismic Design Method (adopted)
1985	Mexico Earthquake	
1989	(Heisei Era)	
1995	Hyogo-ken-nanbu Earthquake	Revision of Building Standard Law (Shape factor)
		Seismic Retrofitting Promotion Law
1999	Great Turkey Earthquake	
	Great Taiwan Earthquake	
2000		Response and Limit Capacity Method
		Housing Performance Display System
2004	Sumatra Earthquake, Indonesia	
2006	Kashmir Earthquake, Pakistan	
2006	Central Java Earthquake, Indonesia	
2008	Wenchuan Earthquake, China	
2010	Haiti Earthquake	
2010	Chile Earthquake	

(2) Shear Walls and Steel Frame Reinforced Concrete Structures

As for the research on earthquake resistant structures, Dr. Tachu Naito succeeded Dr. Toshikata Sano. Dr. Naito paid special attention to the effectiveness of shear walls. He had contrived unique construction “steel frame reinforced concrete” where steel frames are covered with reinforced concrete which is unique to Japan.

Since the damage to brick and masonry buildings was extremely severer than to other buildings in the Great Kanto Earthquake, brick and masonry buildings were hardly constructed since the earthquake. Even modern buildings of steel or reinforced concrete had many problems against earthquake resistant capacity, if their construction practices are similar to those of the east coast of U.S. where there are no earthquakes. It also became evident that shear walls are very effective against earthquakes, especially in the case of the Nippon Kogyo Bank Building designed by Dr. Naito, which suffered only minor damage during the Great Kanto Earthquake. Since then, the steel frame reinforced concrete buildings with shear walls became to be recognized as the most reliable against earthquakes.

(3) Flexibility vs. Rigidity Dispute

After the Great Kanto Earthquake, the research on earthquake resistant technology had greatly progressed. In 1930’s there happened “Flexibility vs. Rigidity Dispute.” It was the dispute whether flexible structures or rigid structures are more reliable against earthquakes. In order to protect buildings against earthquakes, one claimed that buildings should be made stiff and should have shorter natural period of vibration, and another insisted that buildings should be flexible and have longer natural period in order to avoid resonance with earthquake motions.

The representative for Flexible Structure Theory was Dr. Kenzaburo Majima, and the representative for Rigid Structure Theory was Dr. Toshikata Sano and Dr. Kiyoshi Muto. The theory, that is to make natural period longer and to reduce seismic forces which act as inertia forces, has been developed to build high-rise buildings and buildings with base isolation systems. Therefore, at present it is evident that Flexible Structure Theory is superior to Rigid Structure Theory. In 1930’s, however, the characteristics of earthquake motions were not clearly understood, and there were no computers to analyze the behavior of buildings against earthquake motions. Although they did not get any conclusions on the Flexibility vs. Rigidity Dispute, Rigid Structure Theory prevailed after the dispute. The dispute was a starting point of the development of research, e.g. vibration theory and dynamics of structures.

(4) Buildings after the Great Kanto Earthquake

Because of the progress of research after the Great Kanto Earthquake, the Urban Building Law was revised in 1932 to include specifications for mixing of concrete, strength and allowable stress of concrete, and joints of steel members. Namely, welding-joints for steel frames and the rational strength estimation for concrete by introducing water cement ratio became possible.

The Architectural Institute of Japan (AIJ) published the draft “Standard for Structural Calculation of Reinforced Concrete Structures” in 1933, and “Distributing coefficient of horizontal forces” or “D-value method” by Dr. Kiyoshi Muto was included in the draft. Research on steel structures, e.g. welding, continued and AIJ published the draft “Standard for Structural Calculation of Steel Structures” in 1941.

4) After the End of World War II in 1945

(1) Building Standard Law after World War II

Since the Urban Building Law, one allowable stress had been specified for each structural material. But the damage of buildings caused by the Muroto Typhoon in 1934 enlightened the problem of one allowable stress system. After the DIN Standard of Germany introduced the concept of ordinary state and extraordinary state, the specifications for the Urban Building Law was revised in 1937. In order to make use of building materials during World War II, Wartime Temporary Standard was enforced in 1943 and the concept of permanent (long term) and temporary (short term) loading was introduced.

After World War II, the Wartime Temporary Standard was revised to Building Standard 3001 in 1947. Then the Building Standard Law replaced the old regulations in 1950. The concept of “permanent” and “temporary” was introduced to load combinations and allowable stresses. Since the temporary allowable stress became twice of the old allowable stress (equivalent to the permanent allowable stress), the horizontal seismic coefficient became 0.2 which is twice of the old regulations.

As for wooden buildings, the Urban Building Law only stipulated that “Braces or knee braces shall be installed suitably,” but the Building Standard Law Enforcement Order prescribed the size and the necessary amount of braces against seismic forces.

(2) Strong Motion Accelerometers and Recorded Ground Motions

It is important to know the characteristics of earthquake ground motions for earthquake resistant design. Dr. Kyoji Suehiro, the Director of Earthquake Research Institute, the University of Tokyo emphasized the development of strong motion accelerometers and the necessity of strong earthquake motion records, but his opinion was not accepted in Japan. In 1932, he made a speech on the necessity of the strong motion accelerometers and records in the U.S. Then U.S. researchers recognized his opinion immediately, developed strong motion accelerometers, and began to try to record strong earthquake motions in California. A strong earthquake motion (maximum acceleration 0.32g) was recorded during the 1933 Long Beach Earthquake (M=6.8). In 1932, Dr. Biot proposed the concept of response spectrum that is still used for seismic design or in seismic codes.

A famous strong earthquake motion record was obtained at El Centro, California during the 1940 Imperial Valley Earthquake (M=7.1). Incidentally, the El Centro record is still used for dynamic analyses of high-rise buildings and other structures, and it is treated as one of standard earthquake ground motions.

In Japan, it was after World War II that a research on the analysis of earthquake ground motion records began, and the first strong motion accelerometer was installed in a building in 1953.

(3) Dynamic Analyses and High-rise Buildings

It is impossible to analyze earthquake motion records or to calculate structural response caused by earthquake motions by hand calculation. Fortunately electronic computers were being developed. As a result of earthquake response analyses, it was found out that the acceleration response of buildings with longer natural period becomes smaller. Then it was shown that high-rise buildings can also be constructed in Japan.

The records of strong earthquake motions were gradually accumulated since then. With the advancement of computers, the dynamic analyses of the building switched over from the research level to the practical use level.

In 1963, the maximum height limitation of 31 meter since the Urban Building Law was abolished. In 1968, the first high-rise Kasumigaseki Building (147 meters in height, Fig.1.15) in Japan was completed. Since then, high-rise building boom began.

Kasumigaseki Building,
36 story, 147m high

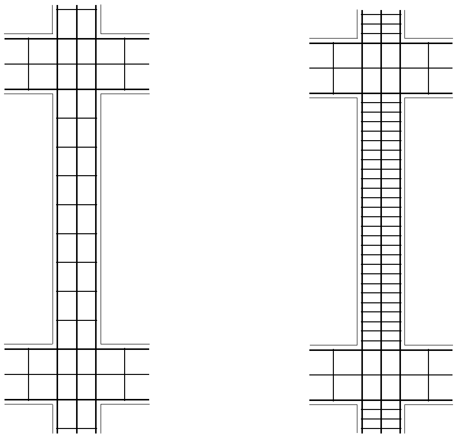
Figure 1.15: First high-rise building in Japan

(4) Tokachi-oki Earthquake and Strengthening of Shear Reinforcement

It was found out from the results of earthquake response analysis that earthquake forces, which act on buildings during severe earthquake motions, are much larger than the value specified by the Building Standard Law. It was also found out the fact that most buildings did not collapse by severe earthquake motions because buildings have overstrength and ductility which are not considered in the calculations.

The 1964 Niigata Earthquake (M=7.5, 26 deaths, 1,960 buildings of total collapse, 6,649 of partial collapse, flooding 15,298) caused severe damage with sinking and tilting of buildings, because of liquefaction of saturated sandy soil. In the 1968 Tokachi-oki Earthquake (M=7.9, 52 deaths, 300 injuries, 673 buildings of total collapse, 3,004 of partial collapse), many reinforced concrete buildings suffered severe damage which had been believed to have enough earthquake resistant capacity.

Then, the Building Standard Law Enforcement Order was revised and shear reinforcement of reinforced concrete columns was strengthened (Fig.1.16). Incidentally, the effectiveness of this revision was proven in the 1995 Hyogo-ken-nanbu Earthquake.



Hoop spacing was 30 cm until 1970,
and 10 cm after 1971.

a) Until 1970 b) After 1971

Figure 1.16: Strengthening of shear reinforcement for RC columns

(5) Seismic Diagnoses

Reinforced concrete buildings which had been designed to be earthquake resistant, suffered unexpected damage in the 1948 Fukui Earthquake (M=7.1, 3,769 deaths, 36,184 buidings of total collapse, 11,816 of partial collapse, 3,851 were destroyed by fire), the 1964 Niigata Earthquake, the 1968 Tokachi-oki Earthquake, the 1975 Ohita-ken-chubu Earthquake (M=6.4, 22 injuries, 58 buildings of total collapse, 93 of partial collapse), etc. Then the seismic diagnosis for reinforced concrete buildings was developed in 1977. This seismic diagnosis was applied to many buildings in Shizuoka Prefecture, Japan and also to buildings in Mexico City after the 1985 Mexico Earthquake (M=8.1, more than 9,500 deaths, more than 400 buildings collapsed). The seismic diagnoses for steel buidings and for wooden buildings were developed in 1979, and the seismic diagnosis for steel frame reinforced concrete buildings in 1986. These diagnoses have been revised considering related research and earthquake damage.

In the Seismic Retrofitting Promotion Law for existing buildings that was established after the Hyogo-ken-nanbu Earthquake, buildings are inspected using these seismic diagnoses.

(6) New Seismic Design Method

The fundamental revision of the seismic design method became necessary because of the development of the earthquake response analysis technology and the damage caused by the 1968 Tokachi-oki Earthquake.

After the 1971 San Fernando Earthquake (M=6.4), a five year national research project for establishing a new seismic design method was carried out from 1972 to 1977 in Japan by the Ministry of Construction, with the cooperation of the Building Research Institute, Public Works Research Institute, universities, private companies and many other organizations. Then the new seismic design method was proposed in 1977.

The 1978 Miyagi-ken-oki Earthquake (M=7.4, 28 deaths, 1,325 injuries, 1,183 buildings of total collapse, 5,574 of partial collapse, Fig.1.17) hit the Sendai area and its occurrence accelerated the adoption of the new seismic design method. The new seismic design method, which had been already proposed, was reviewed and evaluated for use as a practical design method. Then the Building Standard Law Enforcement Order was revised in 1980, and the new seismic design method has been used since 1981.

(Until 1970, the hoop spacing of RC columns was 30 cm, cf. Fig.1.16. Therefore, this column was damaged during 1978 Miyagi-ken-oki Earthquake in spite of good quality of concrete.)

Figure 1.17: Dameged RC column because of poor transverse reinforcement

The new seismic design method was developed introducing up-to-date knowledge of earthquake engineering at that time. The major changes were the introduction of two levels of earthquake motions (severe earthquake motions and moderate earthquake motions), single formula to evaluate seismic forces for buildings with shorter natural period as well as longer natural period, seismic shear coefficient instead of seismic coefficient, consideration of structural balance in plan and in elevation (story drift, shape factor to consider stiffness and eccentricity), structural characteristic factor to consider ductility, etc. The validity of this revision has been proven in the Hyogo-ken-nanbu Earthquake.

(7) The Hyogo-ken-nanbu Earthquake and Countermeasures

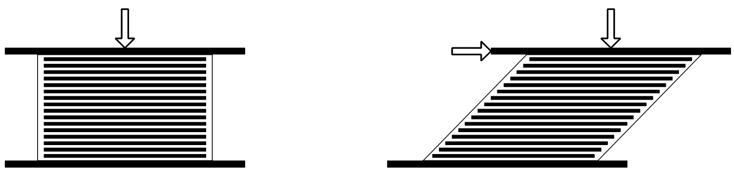
The 1995 Hyogo-ken-nanbu Earthquake (M=7.2, 6,430 deaths, 3 missing, more than 40,000 injuries, more than 240,000 buildings of total collapse or partial collapse, more than 6,000 were destroyed by fire) caused extensive damage that most researchers and engineers had not expected. Until the Hyogo-ken-nanbu Earthquake, Japanese believed that most buildings were safe against earthquakes, even against severe earthquakes like the Great Kanto Earthquake, because the Building Standard Law was strict enough. The Hyogo-ken-nanbu Earthquake, therefore, forced the researchers to reconsider the intensity of severe earthquake motions and the damage caused by earthquakes. It also became an opportunity to introduce performance-based design and a system to display the quality of houses.

In addition, the damage to buildings after the new seismic design method was not so severe as the damage to buildings before, then the seismic diagnoses and seismic retrofitting were applied to existing buildings that were built before the new seismic design method. The Seismic Retrofitting Promotion Law for existing buildings has been enforced since December 1995. The damage to buildings constructed after the new seismic design method was mainly the damage to the soft and weak first story that had been used for parking area or shops. Therefore, the shape factor was revised after the earthquake.

Although the validity of the new seismic design method has been proven in the Hyogo-ken-nanbu Earthquake, the Building Standard Law was revised in 1998. The revision was to make minor amendments to the new seismic design method, and to introduce performance-based response and limit capacity method (see Route $\boxed{4}$ on p.24).

(8) Base Isolation and Response Control

As for base isolation and response control systems, various ideas have been proposed and some of them have been applied to actual buildings. One of the systems that is applied not only for experimental but also practical use is the rubber bearing. The rubber bearing is a device in which thin rubber sheets and steel plates are laminated alternately (Fig.1.18). Its horizontal stiffness is low but the vertical stiffness is high. The device was developed in France in the 1970's. In the late 1970's, it was applied to buildings and nuclear power plants in France, South Africa, New Zealand (Fig.1.19), U.S., etc. In Japan it was used in a house in 1983. After that, it was further developed in technical research institutes of major construction firms in Japan, and the base isolation system (Fig.1.20) gradually spread. The effectiveness of the base isolation, although not in the most severely affected area, was proven in the Hyogo-ken-nanbu Earthquake. Then buildings with base isolation bearings became popular and now there are approximately 1,500 buildings and 3,000 houses with this system in Japan.



a) Stiff against vertical force b) Flexible againt horizontal force

Figure 1.18: Rubber bearing for base isolation
(Rubber sheets and inner steel plates are laminated alternately, and the shape is round or square.)

William Clayton Building in Wellington, New Zealand

Figure 1.19: First large scale building with base isolation

Figure 1.20: Rubber bearing, steel damper and lead damper
(Both isolators and dampers are usually installed in base isolated buildings.)

Besides the base isolation system of rubber bearings, there are several other systems, e.g. sliding or roller bearing systems. But most of them are still for experimental use. There is an active response control system that detects earthquake motions and exerts force to reduce the motion of buildings. A hybrid control system, which combines passive and active response control systems is also proposed. It may take a little more time before these new systems can be applied for practical use.

5) Expectation to Future Development

As described in the previous sections, major changes in seismic codes were made after the damage caused by severe earthquakes. However, in spite of being often struck by severe earthquakes, no measures against earthquakes were adopted before the Meiji Era. Research on seismology and earthquake engineering that started at the end of the 19th century had been developed in the 20th century.

From now on, further developments in the field of seismology (e.g. precise earthquake prediction technique, estimation of earthquake motions, etc.) and earthquake engineering (e.g. performance-based seismic design taking into account the damage extent caused by different levels of earthquake motions, base isolation and response control systems, etc.) are expected.

During the last ten years of the 20th century, the 1994 Northridge Earthquake, U.S.A., the 1995 Hyogo-ken-nanbu (Kobe) Earthquake, Japan, the 1999 Great Turkey Earthquake (M=7.4, more than 17,000 deaths, more than 43,000 injuries, 145,000 buildings of total or partial collapse), and the 1999 Great Taiwan Earthquake (M=7.7, more than 2,200 deaths, more than 8,000 injuries, 17,000 buildings of total or partial collapse) occurred and caused severe and extensive damage.

In the 21st century, seismic events occur more frequently in the world, i.e. the 2004 Sumatra Earthquake, Indonesia (M=9.3, more than 283,000 deaths), the 2006 Kashmir Earthquake, Pakistan (M=7.6, more than 80,000 deaths), the 2006 Central Java Earthquake, Indonesia (M=6.3, more than 5,000 deaths), the 2008 Wenchuan Earthquake, China (M=7.9, more than 69,000 deaths), the 2009 Western Sumatra Earthquake, Indonesia (M=7.6, more than 1,000 deaths), the 2010 Haiti Earthquake (M=7.0, more than 220,000 deaths), the 2010 Chile Earthquake (M=8.8, more than 500 deaths), etc. In this 21st century, we are still not free from earthquake damage, but hopefully the fundamental objective of seismic design, that is to protect human lives and minimize the damage to properties, will be accomplished.

Chapter 2

Seismic Code for Buildings in Japan

2.1 Introduction

After the 1923 Great Kanto Earthquake, the provision that “the horizontal seismic coefficient shall be at least 0.1” was added in 1924 to the Urban Building Law which had been enforced since 1919. It was the first seismic code in the world to introduce the seismic coefficient. The seismic coefficient was specified to be 0.1, because the seismic coefficient of ground surface at Tokyo was estimated to be 0.3 of the acceleration due to gravity during the Great Kanto Earthquake, and the safety factor of material strength to allowable stress was assumed to be 3. (The acceleration response of the building, however, had not been considered.)

Since the enforcement of the Urban Building Law, one allowable stress had been specified for each structural material. After World War II, the Building Standard Law replaced the old regulations in 1950. Then the concept of “permanent” (long term) and “temporary” (short term) was introduced to load combinations and allowable stresses. Since the temporary allowable stress became twice of the old allowable stress (equivalent to the permanent allowable stress), the horizontal seismic coefficient became 0.2 which is twice of the old regulations.

The fundamental revision of the seismic design method became necessary because of the development of the earthquake response analysis technology and the damage caused by the 1968 Tokachi-oki Earthquake and other earthquakes. A five year national research project for establishing a new seismic design method was carried out from 1972 to 1977 in Japan. The 1978 Miyagi-ken-oki Earthquake hit the Sendai area and its occurrence accelerated the adoption of the new seismic design method. The new seismic design method, which had been already proposed, was reviewed and evaluated for use as a practical design method. The Building Standard Law Enforcement Order was revised in 1980, and the new seismic design method has been used since 1981.

The new seismic design method was developed introducing up-to-date knowledge of earthquake engineering at that time. The major changes were the introduction of two levels of earthquake motions (severe and moderate earthquake motions), single formula to evaluate seismic forces for buildings with short natural period as well as with longer natural period, seismic shear coefficient instead of seismic coefficient, consideration of structural balance in plan and in elevation (story drift and shape factor to consider stiffness and eccentricity), structural characteristic factor to consider strength and ductility, etc. The new seismic design method for buildings is now called as “Allowable Stress and Lateral Shear Capacity Method” and it is summarized as

Routes $\boxed{1}$, $\boxed{2}$ and $\boxed{3}$ in the next section.

Although the validity of the new seismic design method has been proven during the 1995 Hyogo-ken-nanbu (Kobe) Earthquake, the Building Standard Law was revised again in 1998 after the earthquake. The revision was to make minor amendments to the new seismic design method, and to introduce performance-based "Response and Limit Capacity Method". This method is included as Route $\boxed{4}$ in the next section.

2.2 Structural Requirments and Design Routes

(1) General

The purpose of seismic design is that buildings should withstand moderate earthquake (rare earthquake) motions, which would occasionally occur during the service life of the buildings with almost no damage, and should not collapse nor harm human lives during severe earthquake (extremely rare earthquake) motions, which would rarely occur during the service life of the buildings.

In order to fulfill the purpose of the seismic design, buildings shall satisfy one or more of the structural requirements specified in the next section, according to the type of the structural systems, floor area, height, etc. Each sequence of structural requirements forms a design route as shown in Table 2.1 and Fig.2.1.

Table 2.1: Structural requirements and seismic design routes

Buildings		Structural requirements	Design route
A	Small scale	(a)	Route $\boxed{0}$
B	Medium scale	(a), (b), (c)	Route $\boxed{1}$
C	Large scale (height ≦ 31 m)	(a), (b), (d), (e), (f)	Route $\boxed{2}$
D	Large scale (31 m< height ≦ 60 m)	(a)*, (b), (d), (g)	Route $\boxed{3}$
A〜D	Any scale (height ≦ 60 m)	(a)**, (d), (h), (i)	Route $\boxed{4}$
E	High-rise (height > 60 m)	(a)**, (j)	Route $\boxed{5}$

* Some structural specifications can be alleviated.

** Structural specifications except for duarabiliry can be alleviated.

(2) Structural requirements

(a) Structural specifications shall be fulfilled for any buildings.

(Some of them can be alleviated for Route $\boxed{3}$, and most of them except for duarability can be alleviated for Routes $\boxed{4}$ and $\boxed{5}$.)

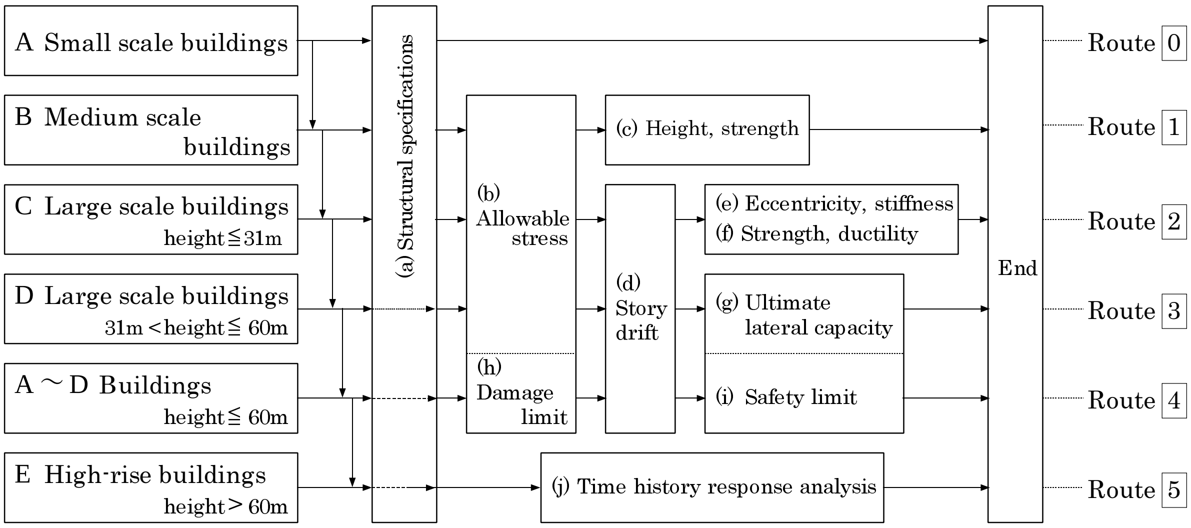


Figure 2.1: Design routes for buildings

(b) Allowable stress calculation is to verify that the stress caused by the lateral seismic shear for moderate earthquake motions shall not exceed the allowable stress for temporary loads.

(This is applied to Routes $\boxed{1}$, $\boxed{2}$ and $\boxed{3}$.)

(c) Height, strength, etc. limitations (along with allowable stress calculation and structural specifications) are to realize seismic safety for small scale buildings.

(This is applied to Route $\boxed{1}$.)

(d) Story drift limitation is to prevent non-structural elements from earthquake damage†.

(This is applied to Routes $\boxed{2}$, $\boxed{3}$ and $\boxed{4}$.)

(e) Stiffness and eccentricity limitation is to prevent earthquake damage concentration to a part of buildings.

(This is applied to Route $\boxed{2}$.)

(f) Strength and ductility limitation is to give adequate strength and ductility against severe earthquake motions.

(This is applied to Route $\boxed{2}$.)

(g) Ultimate lateral capacity limitation is to verify seismic performance against severe earthquake motions, calculating ultimate lateral capacity for each story.

(This is applied to Route $\boxed{3}$.)

†There is another provision, i.e. “Roof tiles, exterior finishings and exterior walls shall not be damaged by wind pressure, earthquake motoins and other vibrations or shocks.”

(h) Damage limit is to verify that the damage shall be within the allowable level against rare earthquake motions.

(This is applied to Route $\boxed{4}$.)

(i) Safety limit is to verify that the building may not collapse against very rare earthquake motions.

(This is applied to Route $\boxed{4}$.)

(j) Time history response analysis is to verify the seismic safety through time history response analysis.

(This is applied to Route $\boxed{5}$.)

(3) Seismic design routes

Route $\boxed{\mathbf{0}}$ requires no structural calculation. The seismic safety is realized by (a) structural specifictions.

(This route is applied to small scale buildings.)

Route $\boxed{\mathbf{1}}$ requires (b) allowable stress calculation, and some limitations of (c) height, strength, etc. Then the seismic safety is realized with the supplements of (a) structural specifications.

(This route is applied to small and medium scale buildings.)

Route $\boxed{\mathbf{2}}$ requires (b) allowable stress calculation, and limitations of (d) story drift, (e) eccentricity and stiffness, and (f) strength and ductility. Then the seismic safety is realized with the supplements of (a) structural specifications.

(This route is applied to buildings, whose height does not exceed 31 m.)

Route $\boxed{\mathbf{3}}$ requires (b) allowable stress calculation, (d) story drift limitation, and (g) ultimate lateral capacity calculation for each story. Then the seismic safety is realized with the supplements of (a) structural specifications (some of them can be alleviated).

(This route is applied to buildings, whose height does not exceed 60 m.)

Route $\boxed{\mathbf{4}}$ requires verifications for (h) damage limit, and (i) safety limit (this route is called "response and limit capacity method"). Then the seismic safety is realized, and (a) structural specifications can be alleviated except for durability.

(This route is applied to buildings, whose height does not exceed 60 m.)

Route $\boxed{\mathbf{5}}$ requires (j) time history response analysis to verify the seismic safety, and (a) structural specifications can be alleviated except for durability.

(This route is applied to all buildings, including whose height exceeds 60 m.)

2.3 Details of Structural Requirements

(a) Structural specifications (Routes $\boxed{0}$〜$\boxed{5}$)

Buildings shall meet the relevant structural specifications specified by the Building Standard Law Enforcement Order, Notifications of the Ministry of Land, Infrastructure, Transport and Tourism (MLIT), the Specifications of the Architectural Institute of Japan (AIJ), etc.

Some specifications may not be applied for Route $\boxed{3}$, and the specifications except for duarability can be alleviated for Routes $\boxed{4}$ and $\boxed{5}$.

(b) Allowable stresses (Routes $\boxed{1}$, $\boxed{2}$ and $\boxed{3}$)

The stresses caused by the lateral seismic shear for moderate earthquake motions, prescribed in Sec.2.4 (1) Lateral Seismic Shear above the Ground Level (p.35) and (4) Lateral Seismic Shear of the Basement (p.41), shall not exceed the temporary allowable stresses.

Load combinatins including the normal, snow and wind conditions are shown in Table 2.2.

Table 2.2: Load combinations

Stress type	Load condition	Ordinary district	Heavy snow district
Permanent	Normal	$G + P$	$G + P$
	Snow		$G + P + 0.7\,S$
Temporary or Damage Limit	Snow	$G + P + S$	$G + P + S$
	Wind	$G + P + W$	$G + P^{*} + W$
			$G + P^{*} + 0.35\,S + W$
	Earthquake	$G + P + K$	$G + P + 0.35\,S + K$
Safety limit	Snow	$G + P + 1.4S$	$G + P + 1.4S$
	Wind	$G + P + 1.6W$	$G + P^{*} + 1.6W$
			$G + P^{*} + 0.35\,S + 1.6W$
Permanent allowable stress is used to verify permanent stress, temporary allowable stress to verify temporary stress and damage limit, and material strength to verify safety limit.	G: stress caused by dead load P: stress caused by live load S: stress caused by snow load W: stress caused by wind load K: stress caused by seismic load		P^{*} Stress caused by the live load shall be reduced appropriately for verification of overturning of buildings and pulling-out of columns.

(c) Height, strength, etc. (Route $\boxed{1}$)

(1) Wooden construction (Route $\boxed{1}$)

Route $\boxed{1}$ is applied to buildings of conventional wooden construction (except the basement) that meet the conditions as follows.

Maximum height ≦ 13 m, and the eaves height ≦ 9 m

(2) Steel construction (Route $\boxed{1}$)

Route $\boxed{\textbf{1-1}}$ is applied to buildings of steel construction (except the basement) that meet all items as follows.

1) Number of stories above the ground level ≦ 3
2) Maximum height ≦ 13 m, and the eaves height ≦ 9 m
3) Maximum span of beams ≦ 6 m
4) Total floor area ≦ 500 m^2
5) Stresses caused by the lateral seismic shear in which the standard shear coefficient C_0 in Eq.(2.16) on p.35 becomes 0.3 do not exceed the temporary allowable stresses.
6) Ends and joints of braces do not break when the braces come to yield (see Fig.2.2).

Joints of braces should not break before the brace come to yield.

Figure 2.2: Damage to brace joint (1978 Miyagi-ken-oki earthquake)

Route $\boxed{\textbf{1-2}}$ is applied to buildings of steel construction (except the basement) that meet all items as follows.

1) Number of stories above the ground level ≦ 2
2) Maximum height ≦ 13 m, and the eaves height ≦ 9 m
3) Maximum span of beams ≦ 12 m
4) Total floor area ≦ 500 m^2
5) Items 5) and 6) of Route $\boxed{1\text{-}1}$ are satisfied ,
6) Stiffness eccentricity ratio† ≦ 0.15
7) Columns, beams and their joints do not loose their strength suddenly because of local buckling or failure, and the connections between the bottom ends of columns and the foundation do not loose their strength suddenly because of anchor bolts failure or foundation collapse.

†See Eq.(2.5) on p.29.

(3) Reinforced and steel frame reinforced concrete construction (Route $\boxed{1}$)

Route $\boxed{1}$ is applied to buildings of reinforced and steel frame reinforced concrete construction (except the basement) that meet all items as follows.

1) Maximum height $\leqq$ 20 m
2) Each story meets the following condition in the longitudinal and transverse directions. The factor "0.7" in the equation is replaced by "1.0" for steel frame reinforced concrete construction.

$$\sum 2.5\,\alpha\,A_w + \sum 0.7\,\alpha\,A_c \geqq Z\,W_i\,A_i \tag{2.1}$$

   where, α is the adjusting factor depending on the concrete strength F_c (N/mm^2) as follows.

$$\alpha = \sqrt{F_c/18} \qquad \text{where} \qquad 1 \leqq \alpha \leqq \sqrt{2} \tag{2.2}$$

   A_w is the horizontal cross-sectional area (mm^2) of reinforced concrete shear walls in the direction concerned, A_c is the cross-sectional area (mm^2) of reinforced concrete columns, and horizontal cross-sectional area (mm^2) of reinforced concrete walls except shear walls in the direction concerned, Z is the seismic hazard zoning factor as shown in Fig.2.12 (p.36), A_i is the lateral shear distribution factor of the i-th story given by Eq.(2.17) on p.36 and as shown in Fig.2.14 (p.37), W_i is the weight (N) of the building above the i-th story.
3) The design shear Q_D in the following eqaution does not exceed the temporary allowable shear.

$$Q_D = \min\left[(Q_L + n\,Q_E), (Q_0 + Q_y)\right] \tag{2.3}$$

   where, Q_L is the shear caused by permanent loads (that can be neglected for columns), n shall be equal to or greater than 1.5 (2.0 for shear walls) and 1.0 for steel frame reinforced concrete construction, Q_E is the shear caused by the moderate earthquake motions, Q_0 is the shear caused by permanent loads assuming that the both ends are simply supported (that can be neglected for columns), Q_y is the shear assuming that the both ends yield in bending (for column top end the effect of the beams that yield in bending can be included) (see Fig.2.3 and Fig.2.4).

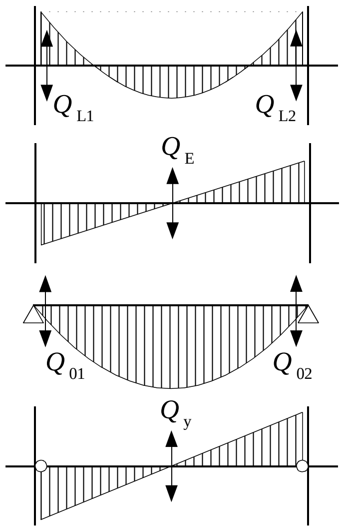
(4) Masonry construction (Route $\boxed{1}$)

Route $\boxed{1}$ is applied to buildings of masonry construction (except the basement) that meet the condition as follows.

The number of stories above the ground level $\leqq$ 3

(d) Story drift (Routes $\boxed{2}$, $\boxed{3}$ and $\boxed{4}$)

The drift of each story (except the basement) of the building caused by the lateral seismic shear for moderate earthquake motions prescribed in Sec.2.4 (1) (p.35) or by the lateral seismic force at damage limit in Sec.2.4 (2) (p.38) shall not exceed 1/200 of the story height. This value can be increased to 1/120, if the nonstructural elements shall have no severe damage at the increased story drift limitation.



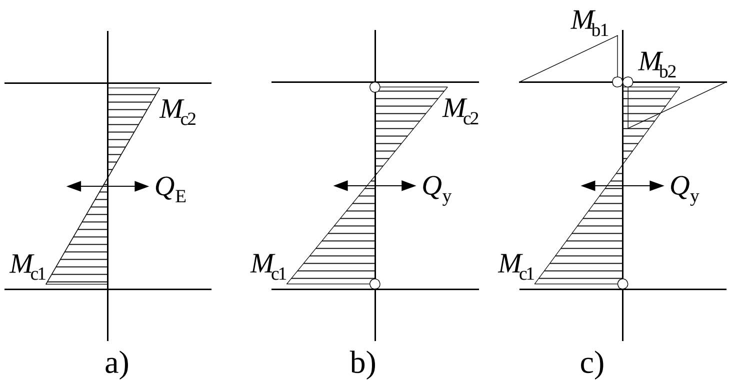
a) Q_L is the shear caused by permanent loads.

b) Q_E is the shear caused by moderate earthquake motions.

c) Q_0 is the shear caused by permanent loads assuming both ends are simply supported.

d) Q_y is the shear assuming both ends yield in bending.

Figure 2.3: Three kinds of shear of the beam



a) Q_E is the shear caused by moderate earthquake motions

b) Q_y is the shear assuming both ends of the column yield in bending

c) Q_y is the shear assuming beam ends yield in bending at column top and column bottom end yields in bending

Figure 2.4: Three kinds of shear of the column

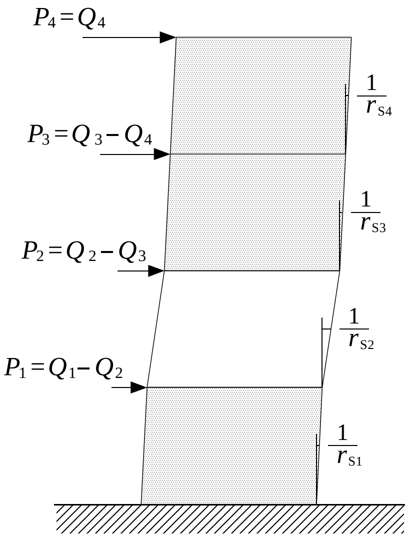
(e) Stiffness and Eccentricity (Route $\boxed{2}$)

(1) Lateral stiffness ratio (Route $\boxed{2}$)

The following lateral stiffness ratio R_s of each story (except the basement) shall be equal to or greater than 0.6 (see Fig.2.5).

$$R_s = \frac{r_s}{\bar{r}_s} \tag{2.4}$$

where, r_s is the lateral stiffness, which is defined as the story height divided by the story drift caused by the lateral seismic shear for moderate earthquake motions prescribed in Sec.2.4(1) on p.35, and $\bar{r}_s$ is the mean lateral stiffness that is defined as the arithmetic mean of r_s's.



$$\bar{r}_s = \frac{\sum_{i=1}^{n} r_{si}}{n}$$

$$R_s = \frac{r_{si}}{\bar{r}_s}$$

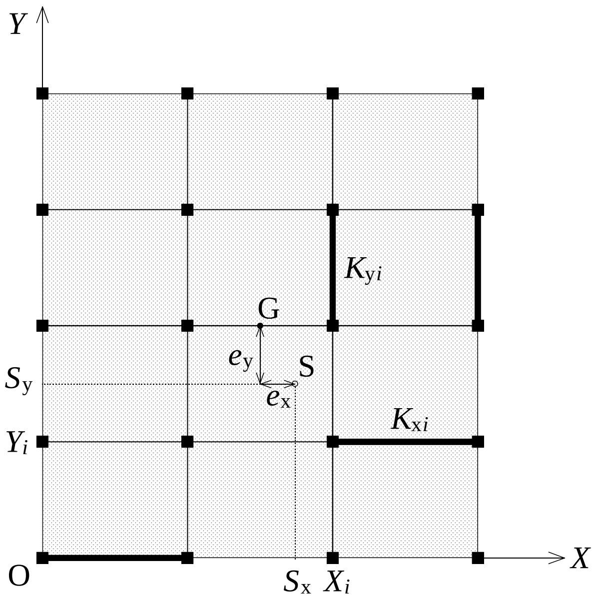
Figure 2.5: Lateral stiffness ratio R_s of the i-th story

(2) Stiffness eccentricity ratio (Route $\boxed{2}$)

The following stiffness eccentricity ratio R_e of each story (except the basement) shall be equal to or less than 0.15 (see Fig.2.6).

$$R_e = \frac{e}{r_e} \tag{2.5}$$

where, e is the eccentricity of the center of stiffness from the center of mass, and r_e is the elastic radius that is defined as the square root of the torsional stiffness divided by the lateral stiffness.



G: center of mass

S: center of stiffness

K_{xi}, K_{yi}: lateral stiffness of element i in X and Y directions

$$S_x = \frac{\sum (K_{yi} X_i)}{\sum K_{yi}} \qquad S_y = \frac{\sum (K_{xi} Y_i)}{\sum K_{xi}}$$

$$K_r = \sum \left\{K_{xi}(Y_i - S_y)^2\right\} + \sum \left\{K_{yi}(X_i - S_x)^2\right\}$$

$$r_{ex} = \sqrt{\frac{K_r}{\sum K_{xi}}} \qquad r_{ey} = \sqrt{\frac{K_r}{\sum K_{yi}}}$$

$$R_{ex} = \frac{e_y}{r_{ex}} \qquad R_{ey} = \frac{e_x}{r_{ey}}$$

Figure 2.6: Stiffness eccentricity ratio R_e in X and Y directions

(f) Strength and ductility (Route $\boxed{2}$)

(1) Wooden construction (Route $\boxed{2}$)

Route $\boxed{2}$ is applied to buildings of wooden construction (except the basement) that meet all items as follows.

1) The lateral seismic shear Q for moderate earthquake motions prescribed in Sec.2.4 (1) (p.35) shall be increased as follows.

$$Q_\mathrm{b} = (1 + 0.7\beta)Q \leqq 1.5Q \tag{2.6}$$

   where, Q_b is the increased lateral seismic shear and β is the ratio of the lateral shear of braces to the total lateral seismic shear of the story.
2) Wooden braces shall not have cleavage or shear failure, when the brace ends or their joints come to yield because of the stress caused by compressive strain.
3) The ends and joints of braces shall not break when the braces other than wood come to yield.
4) The aspect ratio of the building above the groud level $\leqq 4$
5) Columns, beams and their joints shall not loose their strength suddenly due to cleavage or shear failure.

(2) Steel construction (Route $\boxed{2}$)

Route $\boxed{2}$ is applied to buildings of steel construction (except the basement) that meet all items as follows.

1) The lateral seismic shear Q for moderate earthquake motions prescribed in Sec.2.4 (1) (p.35) shall be increased as follows.

$$Q_\mathrm{b} = (1 + 0.7\beta)Q \leqq 1.5Q \tag{2.7}$$

   where, Q_b is the increased lateral seismic shear and β is the ratio of the lateral shear of braces to the total lateral seismic shear of the story.
2) The ends and joints of braces shall not break when the braces come to yield.
3) The width-to-thickness ratio of plate elements of columns and beams shall satisfy the requirements in Table 2.3 in order to prevent local buckling.
4) The aspect ratio of the building above the groud level $\leqq 4$
5) Columns, beams and their joints shall not loose their strength suddenly due to local buckling or failure.

(3) Reinforced concrete or steel frame reinforced concrete construction (Route $\boxed{2}$)

Route $\boxed{\textbf{2-1}}$ is applied to buildings of reinforced concrete or steel frame reinforced concrete construction (except the basement) that meet all items as follows.

Table 2.3: Width-to-thickness ratio of steel members (Route $\boxed{2}$)

Members	Section	Portion	Width-to-thickness ratio
Columns	H	Flange	$9.5\sqrt{235/F}$
		Web	$43\sqrt{235/F}$
	□	–	$33\sqrt{235/F}$
	○	–	$50(235/F)$*
Beams	H	Flange	$9\sqrt{235/F}$
		Web	$60\sqrt{235/F}$

F : standard strength (N/mm^2), where $205 \leqq F \leqq 375$.
* : diameter-to-thickness ratio

1) Each story shall meet the following condition in the longitudinal and transverse directions.

$$\sum 2.5\,\alpha\,A_{\mathrm{w}} + \sum 0.7\,\alpha\,A_{\mathrm{c}} \geqq 0.75\,Z\,A_i\,W_i \tag{2.8}$$

where α, A_{w}, A_{c}, Z, A_i and W_i are the same as in Eq.(2.1). The factor "0.7" is replaced by "1.0" for steel frame reinforced concrete columns.

2) Design shear Q_{D} in the following eqaution does not exceed temporary allowable shear capacity.

$$Q_{\mathrm{D}} = \min\left[(Q_{\mathrm{L}} + n\,Q_{\mathrm{E}}), (Q_0 + Q_{\mathrm{y}})\right] \tag{2.9}$$

where, Q_{L} is the shear caused by permanent loads (that can be neglected for columns), n shall be equal to or greater then 2.0 (for columns with spandrel walls, 2.0 or the story height divided by the opening height which ever the greater, see Fig.2.7), Q_{E} is the shear caused by the moderate earthquake motions, Q_0 is the shear caused by permanent loads assuming that the both ends are simply supported (that can be neglected for columns), Q_{y} is the shear assuming that the both ends of columns or beams reach their flexural capacity, where the flexural capacity of the column top can be one half of the sum of flexural capacities of beams connected to the column top (the sum of flexural capacities of the beams for uppermost columns) if it does not exceed the flexural capacitiy of the column top (see Fig.2.3 and Fig.2.4 on p.28).

3) The aspect ratio of the building above the groud level $\leqq 4$

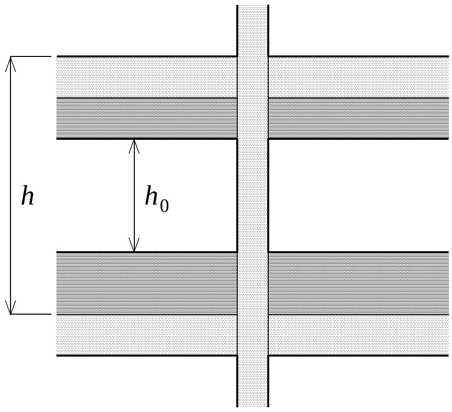
Route $\boxed{\textbf{2-2}}$ is applied to buildings of reinforced concrete or steel frame reinforced concrete construction (except the basement) that meet all items as follows.

1) Each story shall meet the following condition in the longitudinal and transverse directions.

$$\sum 1.8\,\alpha\,A_{\mathrm{w}} + \sum 1.8\,\alpha\,A_{\mathrm{c}} \geqq Z\,A_i\,W_i \tag{2.10}$$

where A_{w}, A_{c}, Z, A_i, W_i and β are the same as in Eq.(2.1). The factor "1.8" is replaced by "2.0" for steel frame reinforced concrete columns and for reinforced concrete shear walls bounded by steel frame reinforced concrete columns.

2) The items 2) and 3) of Route $\boxed{2\text{-}1}$ shall be satisfied.



For columns with spandrel walls, n is the greater of 2.0 or h/h_0, where h is the story height and h_0 is the height of the opening.

Figure 2.7: Column with spandrel walls and n

Route $\boxed{\textbf{2-3}}$ is applied to buildings of reinforced concrete or steel frame reinforced concrete construction (except the basement) that meet all items as follows.

1) When the beam end reaches at its flexural capacity, the adjoining columns and walls to the beam shall not exceed their flexural capacities and shall not have shear failure.
2) The design shear Q_D of columns and beams in the following eqaution shall not exceed the temporary allowable shear capacity.

$$Q_D = Q_0 + n\,Q_u \tag{2.11}$$

where, Q_0 is the shear caused by permanent load assuming that the both ends are simply supported (that can be neglected for columns) (see Fig.2.3 c on p.28), n shall be eqaul to or more than 1.1 (1.0 for the uppermost columns that the top end of the column yields in bending and for the first story columns that the bottom end of the column yields in bending), and Q_u is the shear caused by the conditon of the item 1) (see Fig.2.8).

The design shear Q_D and bending moment M_D of shear walls in the following eqautions shall not exceed the temporary allowable shear capacity and flexural capacity, respectively.

$$Q_D = n_1\,Q_W \quad \text{and} \quad M_D = n_2\,M_W \tag{2.12}$$

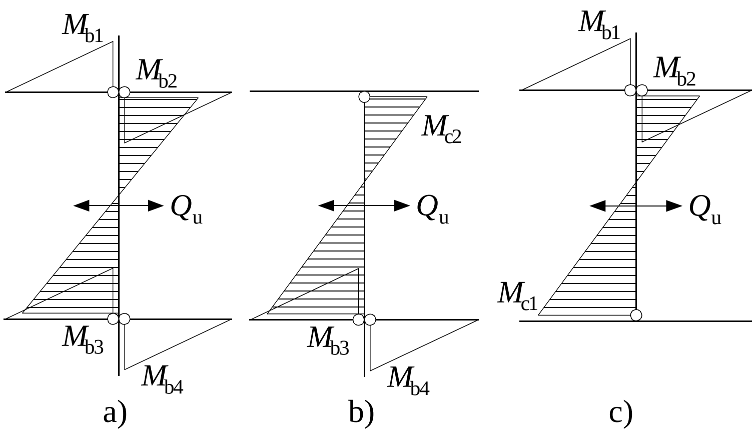
where, n_1 and n_2 shall be equal to or greater than 1.5, Q_W is the shear and M_W is the bending moment when the the building come to global collapse.
3) The aspect ratio of the building above the groud level $\leqq 4$

(g) Ultimate Lateral Capacity (Route $\boxed{3}$)

The computed ultimate lateral capacity (lateral load bearing capacity) of each story (except the basement) shall be equal to or greater than the specified lateral shear Q_{un} determined as follows.

$$Q_{un} = D_s\,F_{es}\,Q_{ud} \tag{2.13}$$

where, Q_{ud} is the lateral seismic shear for severe earthquake motions defined as Q_i in Eq.(2.15) on p.35, D_s is the structural characteristic factor given by Table 2.4, and F_{es} is the shape factor which shall be determined as follows.



a) Shear of the column assuming all adjoining beams yield in bending

b) Shear of the uppermost story column assuming the column top end yields in bending

c) Shear of the first story column assuming the column bottom end yields in bending

Figure 2.8: Shear of the column Q_u

$$F_{es} = F_e F_s \tag{2.14}$$

where, F_e and F_s are given in Table 2.5 and Table 2.6, respectively (see Fig.2.9).

Table 2.4: Structural characteristic factor* D_s

Behavior of members		Type of frame		
		(1) Ductile moment frame	(2) Frame other than listed in (1) & (3)	(3) **
A	Members of excellent ductility	0.3	0.35	0.4
B	Members of good ductility	0.35	0.4	0.45
C	Members of fair ductility	0.4	0.45	0.5
D	Members of poor ductility	0.45	0.5	0.55

* The values can be decreased by 0.05 for steel frame reinforced concrete or steel buildings.

** Frames with shear walls or braces for reinforced concrete and steel frame reinforced concrete buildings, and frames with compressive braces for steel buildings.

(h) Damage Limit (Route $\boxed{4}$)

The lateral seismic shear at damage limit (except the basement) that is evaluated according to Sec.2.4 (2) (i) (p.38) shall not exceed the lateral capacity of the building at damage limit.

Table 2.5: Shape factor F_e and stiffness eccentricity ratio R_e

R_e of Eq.(2.5) on p.29	F_e
$R_e \leqq 0.15$	1.0
$0.15 < R_e < 0.3$	linear interpolation
$0.3 \leqq R_e$	1.5

Table 2.6: Shape factor F_s and lateral stiffness ratio R_s

R_s of Eq.(2.4) on p.28	F_s
$R_s \geqq 0.6$	1.0
$0.6 > R_s$	$2.0 - R_s/0.6$

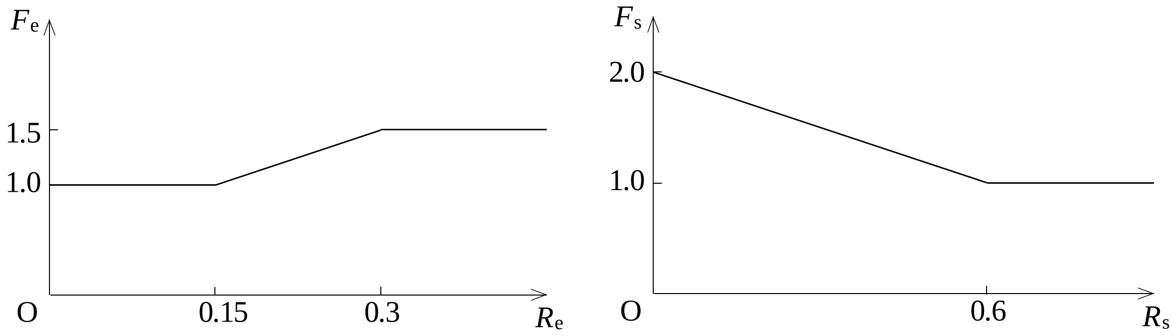


Figure 2.9: Shape factor $F_{es} = F_e\,F_s$

(i) Safety Limit (Route $\boxed{4}$)

The lateral seismic shear at safety limit (except the basement) that is evaluated according to to Sec.2.4 (2) (ii) (p.39) shall not exceed the ultimate lateral capacity of the building.

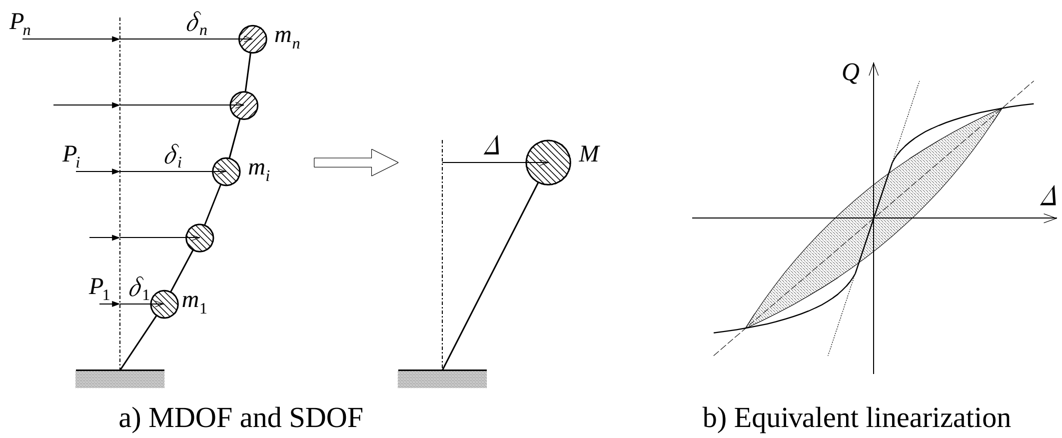


Figure 2.10: Concept of performance-based response and limit capacity method (Route $\boxed{4}$)

(j) Time History Response Analysis (Route $\boxed{5}$)

The building whose height exceeds 60 m shall be confirmed of its structural safety, continuously verifying its capacity and deformation using time history response analysis.

The earthquake motions for the time history response analysis shall be estimated using the acceleration response spectrum in Sec.2.4 (3) (p.40) on the engineering bedrock†.

Site-specific earthquake motions can also be used for the analysis, considering the features of the sources, travel paths and soil conditions of the site.

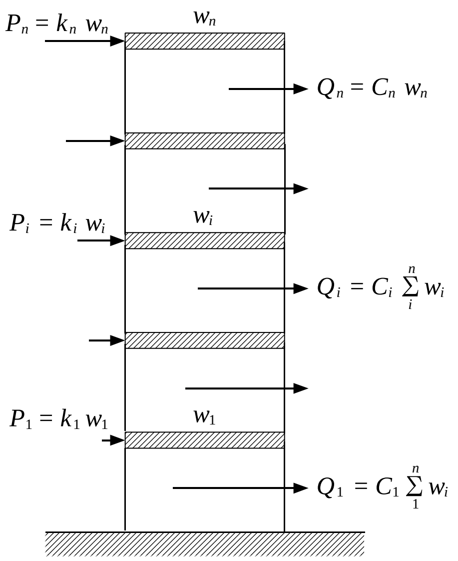
2.4 Seismic Loads and Response Spectra

(1) Lateral Seismic Shears above the Ground Level (Routes $\boxed{1}$, $\boxed{2}$ and $\boxed{3}$)

The lateral seismic shear Q_i of the i-th story above the ground level shall be determined as follows.

$$Q_i = C_i W_i \tag{2.15}$$

where, C_i is the lateral seismic shear coefficient of the i-th story as determined in accordance with Eq.(2.16), and W_i is the weight of the building above the i-th story (see Fig.2.11, where $W_i = \sum_{j=i}^{n} w_j$). The weight of the building shall be the sum of dead load and the applicable portion of live load. In heavy snow districts, the effect of snow load shall be considered (see Table 2.2 on p.25).



P_i: lateral seismic force of the i-th story
Q_i: lateral seismic shear of the i-th story
k_i: lateral seismic coefficient of the i-th story
C_i: lateral seismic shear coefficient of the i-th story
w_i: weight of the i-th story

Figure 2.11: Lateral seismic force P_i and shear Q_i of the i-th story

The lateral seismic shear coefficient C_i of the i-th story shall be determined as follows.

$$C_i = Z R_t A_i C_0 \tag{2.16}$$

†The engineering bedrock is defined as a layer of sufficient thickness whose shear wave velocity is approximately equal to or greater than 400 m/s.

where, Z is the seismic hazard zoning factor (see Fig.2.12), R_{t} is the design spectral factor defined in Table 2.7 (see Fig.2.13), which shall be determined according to the type of soil profile in Table 2.8 and the fundamental natural period T of the building, A_i is the lateral shear distribution factor defined in Eq.(2.17) (see Fig.2.14), and C_0 is the standard shear coefficient, which shall be equal to or greater than 0.2 for moderate earthquake motions and 1.0 for severe earthquake motions.

Standard shear coefficient

Substituting $Z = 1.0$ (for most parts of Japan including Tokyo), $R_t = 1.0$ (for short period buildings) and $A_i = 1.0$ (for the first story of buildings), Eq.(2.16) becomes $C_i = C_0$. Therefore, the standard shear coefficient C_0 indicates the base shear coefficient of low-rise buildings in most parts of Japan.

If we assume that the acceleration amplification factor in the building is 2.5, the standard shear coefficient $C_0 = 0.2$ for moderate earthquake motions divided by 2.5 becomes 0.08. Therefore the ground acceleration for moderate earthquake motions can be estimated 0.08 g, where g is the acceleration due to garvity. And the standard shear coefficient $C_0 = 1.0$ for severe earthquake motions can be interpreted that the ground acceleration for severe earthquake motions is 0.4 g.

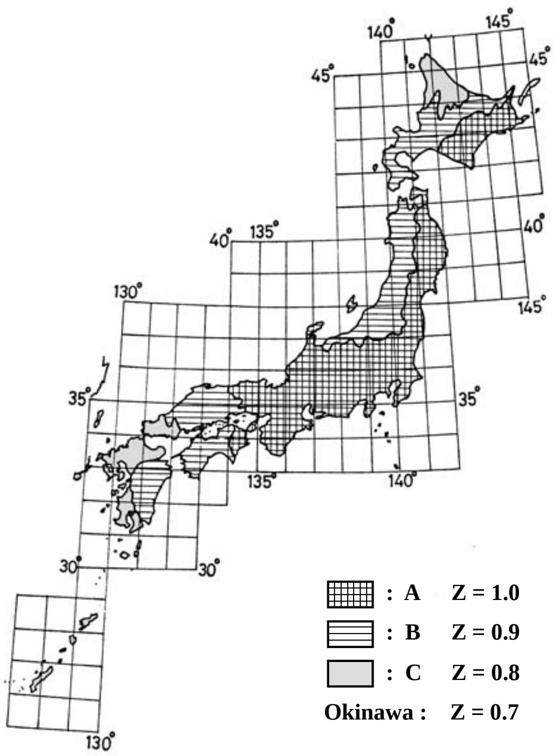


Figure 2.12: Seismic hazard zoning factor Z

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right)\frac{2T}{1 + 3T} \tag{2.17}$$

where, α_i is the normalized weight of the i-th story, which is calculated as the weight above the i-th story divided by the weight above the ground level as follows.

$$\alpha_i = \frac{\sum_{j=i}^{n} w_j}{\sum_{j=1}^{n} w_j} = \frac{W_i}{W} \tag{2.18}$$

where w_j is the weignt of the j-th story.

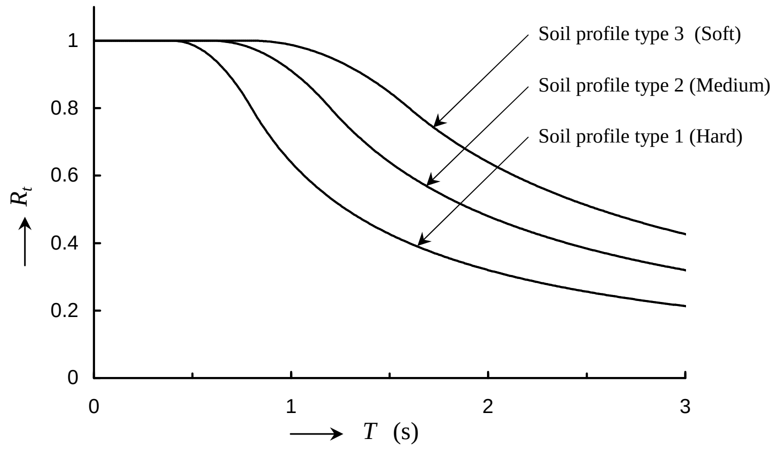


Figure 2.13: Design spectral factor R_t

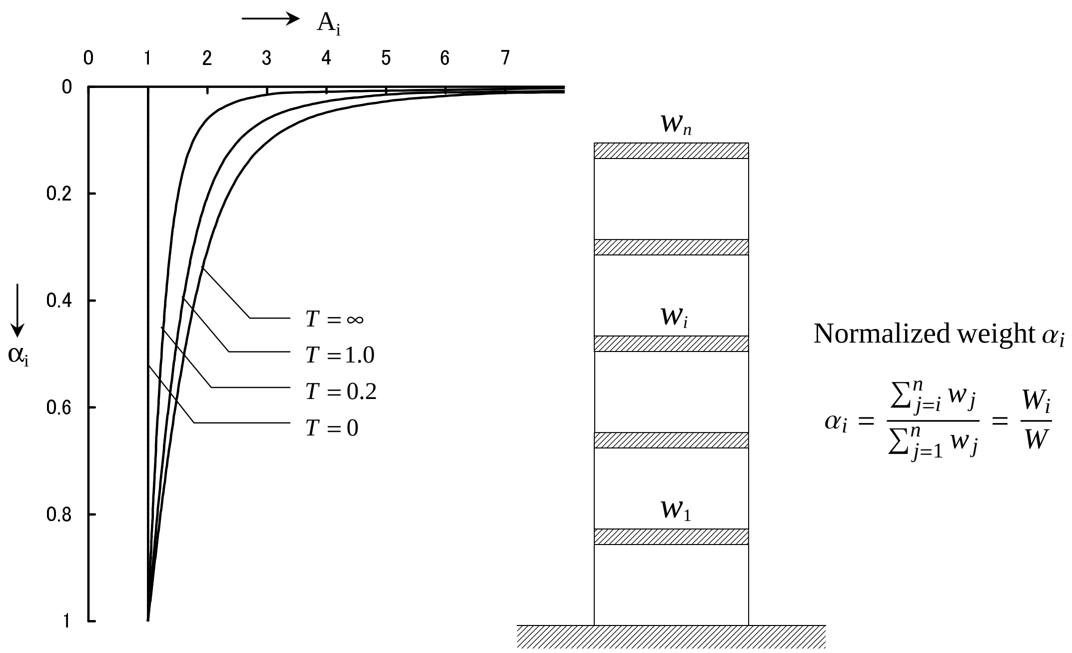


Figure 2.14: Lateral shear distribution factor A_i

The fundamental natural period of the building T (s) can be determined as follows.

$$T = h\,(0.02 + 0.01\lambda) \tag{2.19}$$

where, h (m) is the height of the building, and λ is the ratio of the total height of stories of wooden or steel construction to the height of the building.

The period can also be evaluated by eigenvalue analysis considering the mass and stiffness of the building. The value of R_t, however, can not be less than the value given by Table 2.7.

Table 2.7: Design spectral factor* R_t

T	$T < T_c$	$T_c \leqq T < 2T_c$	$T \geqq 2T_c$
R_t	1	$1 - 0.2\left(\frac{T}{T_c} - 1\right)^2$	$\frac{1.6T_c}{T}$

* : R_t can also be calculated by other methods, but the calculated value shall not be less than 0.75 of the value given by this table.
T : Fundamental natural period (s) of the building.
T_c : Critical period (s) of the soil (see Table 2.8).

Table 2.8: Classification of soil

Soil Profile	Ground Characteristics	T_c (s)
Type 1 (Hard soil)	Ground consisting of rock, hard sandy gravel, etc., classified as tertiary or older, or Ground whose period, estimated by calculation or by other investigation, is equivalent to that of the above.	0.4
Type 2 (Medium soil)	Other than Type 1 or 2.	0.6
Type 3 (Soft soil)	Alluvium consisiting of soft delta deposits, topsoil, mud, or the like (including fills, if any), whose depth is 30 m or more, land obtained by reclamation of marsh, muddy sea bottom, etc., where the depth of the reclaimed ground is 3 m or more and where 30 years have not yet elapsed since the time of reclamation, or Ground whose period, estimated by calculation or by other investigation, is equivalent to that of the above.	0.8

Critical period T_c is used in Table 2.7.

(2) Lateral Seismic Forces above the Ground Level (Route 4)

(i) Damage Limit (Route 4)

The lateral seismic force P_{di} (kN) of the i-th story at damage limit shall be determined as follows.

$$P_{di} = S_{Ad}\, m_i\, B_{di}\, Z\, G_s \tag{2.20}$$

where, S_{Ad} is the acceleration response at damage limit on the engineering bedrock, m_i (t) is the mass of the i-th story, B_{di} is the acceleration distribution factor at damage limit, Z is the seismic hazard zoning factor (see Fig.2.12), and G_s is the surface soil amplification factor.

S_{Ad} (m/s^2) is given as follows (see Fig.2.15).

$$S_{\mathrm{Ad}} = \begin{cases} (0.64 + 6\,T_{\mathrm{d}}) & \text{for } T_{\mathrm{d}} < 0.16 \\ 1.6 & \text{for } 0.16 \leqq T_{\mathrm{d}} < 0.64 \\ 1.024/T_{\mathrm{d}} & \text{for } 0.64 \leqq T_{\mathrm{d}} \end{cases} \tag{2.21}$$

where, T_{d} (s) is the response period of the building at damage limit.

(ii) Safety Limit (Route $\boxed{4}$)

The lateral seismic force P_{si} (kN) of the i-th story at safety limit shall be determined as follows.

$$P_{\mathrm{s}i} = S_{\mathrm{As}}\, m_i\, B_{\mathrm{s}i}\, F_{\zeta}\, Z\, G_{\mathrm{s}} \tag{2.22}$$

where, S_{As} is the acceleration response at safety limit on the engineering bedrock, $B_{\mathrm{s}i}$ is the acceleration distribution factor at safety limit, and F_{ζ} is the acceleration reduction factor due to damping at safety limit.

S_{As} (m/s^2) is given as follows (see Fig.2.15).

$$S_{\mathrm{As}} = \begin{cases} (3.2 + 30\,T_{\mathrm{s}}) & \text{for } T_{\mathrm{s}} < 0.16 \\ 8.0 & \text{for } 0.16 \leqq T_{\mathrm{s}} < 0.64 \\ 5.12/T_{\mathrm{s}} & \text{for } 0.64 \leqq T_{\mathrm{s}} \end{cases} \tag{2.23}$$

where, T_{s} (s) is the response period of the building at safety limit.

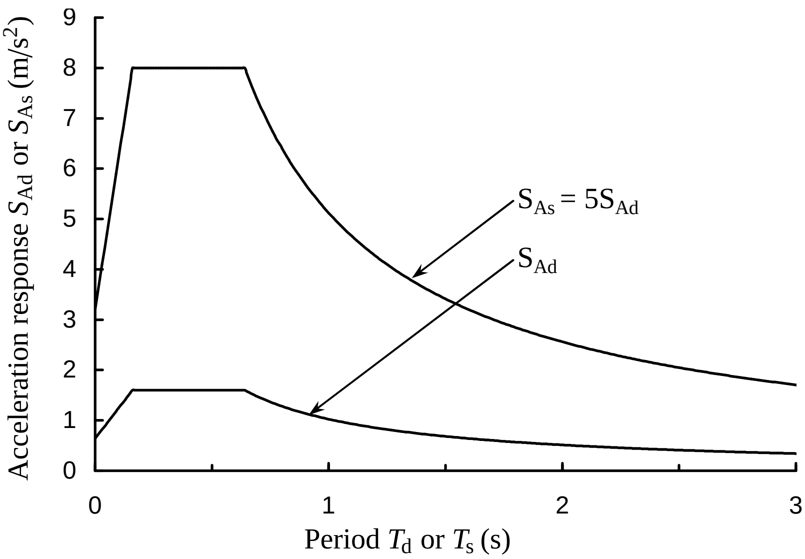


Figure 2.15: Acceleration response spectra at damage and safety limits

Comparison of design spectra of Route $\boxed{3}$ “Allowable Stress and Lateral Shear Capacity Method” and Route $\boxed{4}$ “Response and Limit Capacity Method”.

The curve (a) in Fig.2.16 shows the design spectral factor of soil profile type 2 in Fig.2.13, that can be interpreted as the normalized base shear coefficient (base shear coefficient in g for multi-degree of freedom systems) spectrum for severe earthquake motions of Route $\boxed{3}$. Multiplying the curve (a) by 1.23 for short period (acceleration response constant) range and 1.1 for longe period (velocity response constant) range. the base shear spectrum of the curve (a) is converted into the acceleration response spectrum (for single-degree of freedom systems) of the curve (b). Then, dividing the curve (b) by 1.5 for short period range and by 2 for long period range, the curve (b) is modified into the curve (c) that indicates the acceleration response spectrum at safety limit on the engineering bedrock. It can be seen that the curve (c) is identical to the the acceleration response S_{As} at safety limit in Fig.2.15. Therefore, the design response spectrum for severe earthquake motions of Route $\boxed{3}$ is equivalent to the response spectrum at safety limit of Route $\boxed{4}$.

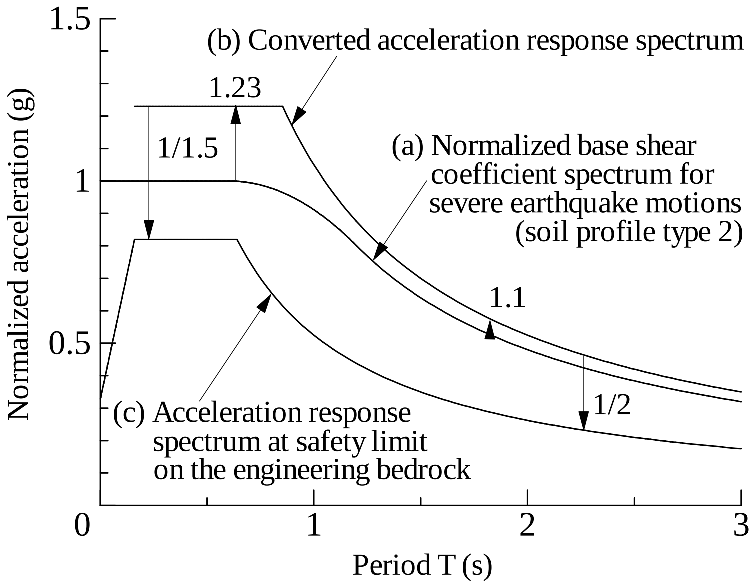


Figure 2.16: Comparison of acceleration response spectra for severe earthquake motions and at safety limit

(3) Acceleration Response Spectrum (Route $\boxed{5}$)

The acceleration response for rare earthquake motions on the engineering bedrock shall be the same as the acceleration response at damage limit of Eq.(2.21), and the acceleration response for extremely rare earthquake motions on the engineering bedrock shall be the same as the acceleration response at safety limit of Eq.(2.23) (see Fig.2.15).

The earthquake motions at the ground level shall be estimated, using the product of the acceleration response at engineering bedrock and the seismic hazard zoning factor Z, and considering the amplification of earthquake motions through surface soil above the engineering bedrock.

(4) Lateral Seismic Shear of the Basement (Routes $\boxed{1}$, $\boxed{2}$, $\boxed{3}$ and $\boxed{4}$)

The lateral seismic shear of the basement Q_B shall be determined as follows (see Fig.2.17 a, where $Q_p = Q_1 + Q_2 + Q_3$).

$$Q_B = Q_p + k\,W_B \tag{2.24}$$

where, Q_p is the portion of the lateral seismic shear of the first story that will extend to the basement, k is the design seismic coefficient of the basement as determined in accordance with Eq.(2.25), and W_B is the weight of the basement (see Fig.2.17 b).

$$k \geqq 0.1\left(1 - \frac{H}{40}\right) Z \tag{2.25}$$

where, H (m) is the depth of the basement. The value of H shall be fixed at 20 m in such cases where the basement depth exceeds 20 m. Z is the seismic hazard zoning factor (see Fig.2.12).

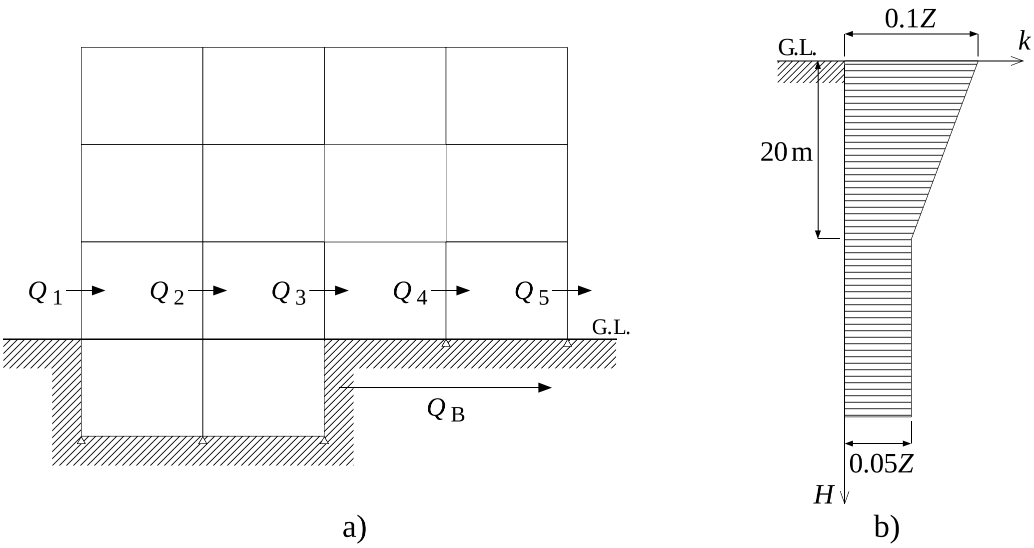


Figure 2.17: Lateral seismic shear Q_B and design seismic coefficient k of the basement

2.5 Parameters for Route $\boxed{4}$

(1) Response Period (Route $\boxed{4}$)

(i) Response Period at Damage Limit (Route $\boxed{4}$)

The response period T_d (s) at damage limit is calculated as follows. The period can be muliplied by the period adjustment factor r of Eq.(2.34). The period can also be evaluated by eigenvalue analysis considering the mass and stiffness of the building.

$$T_d = 2\pi \sqrt{M_{ud} \frac{\Delta_d}{Q_d}} \tag{2.26}$$

where, M_{ud} (t) is the effective mass of the building at damage limit given as follows (see Fig.2.10 a) on p.34).

$$M_{ud} = \frac{(\sum m_i\,\delta_{di})^2}{\sum m_i\,\delta_{di}^2} \tag{2.27}$$

where, m_i (t) is the mass of the i-th story, $\delta_{\mathrm{d}i}$ (m) is the displacement of the i-th story relative to the base caused by the lateral force $P^*_{\mathrm{d}i}$ (kN) at damage limit given as follows.

$$P^*_{\mathrm{d}i} = \frac{B_{\mathrm{d}i}\, m_i}{\sum_{i=1}^{n} B_{\mathrm{d}i}\, m_i} Q_{\mathrm{d}} \tag{2.28}$$

where, $B_{\mathrm{d}i}$ is the acceleration distribution factor of the i-th story at damage limit, n is the number of stories of the building above the ground level, and Q_{d} (kN) is the lateral capacity of the building at damage limit.

Q_{d} is calculated as the product of the weight of the building and the minimum base shear coefficient $q_{\mathrm{d}i}$ that is calculated from the lateral capacity $Q_{\mathrm{d}i}$ of the i-th story at damage limit given as follows.

$$q_{\mathrm{d}i} = \frac{Q_{\mathrm{d}i}}{\dfrac{\sum_{i=i}^{n} B_{\mathrm{d}i}\, m_i}{\sum_{i=1}^{n} B_{\mathrm{d}i}\, m_i} \sum_{i=1}^{n} m_i\, g} \tag{2.29}$$

The equivalent displacement of the building Δ_{d} at damage limit is calculated as follows (see Fig.2.10 a) on p.34).

$$\Delta_{\mathrm{d}} = \frac{\sum m_i\, \delta_{\mathrm{d}i}^2}{\sum m_i\, \delta_{\mathrm{d}i}} \tag{2.30}$$

(ii) Response Period at Safety Limit (Route $\boxed{4}$)

The response period T_{s} (s) at safety limit is calculated in the same manner as to calculated the response period T_{d} (s) at damage limit, replacing the parameters for damage limit T_{d}, M_{ud}, Δ_{d}, Q_{d}, $\delta_{\mathrm{d}i}$, $P^*_{\mathrm{d}i}$, $B_{\mathrm{d}i}$ and $q_{\mathrm{d}i}$ by the parameters for safety limit T_{s}, M_{us}, Δ_{s}, Q_{s}, $\delta_{\mathrm{s}i}$, $P^*_{\mathrm{s}i}$, $B_{\mathrm{s}i}$ and $q_{\mathrm{s}i}$, respectively.

However, Q_{s} is calculated as the product of the weight of the building and the minimum base shear coefficient $q_{\mathrm{s}i}$ that is calculated from the ultimate lateral capacity $Q_{\mathrm{u}i}$ of the i-th story as follows.

$$q_{\mathrm{s}i} = \frac{Q_{\mathrm{u}i}}{F_{\mathrm{e}i} \dfrac{\sum_{i=i}^{n} B_{\mathrm{s}i}\, m_i}{\sum_{i=1}^{n} B_{\mathrm{s}i}\, m_i} \sum_{i=1}^{n} m_i\, g} \tag{2.31}$$

where, $F_{\mathrm{e}i}$ is the shape factor as a function of the stiffness eccentricity ratio of the i-th story given in the same manner as in Eq.(2.14) on p.33 and in Table 2.5 on p.34.

(iii) Displacement at Safety Limit (Route $\boxed{4}$)

The displacement at safety limit is evaluated as the displacement when one of the structural members reaches the limit deformation angle R_{u} (rad) given by Eq.(2.32) (see Fig.2.18). If removing the member that reaches the limit deformation will not result in the collapse of the building, the displacement at safety limit can be calculated from the structure after removing that member.

$$R_{\mathrm{u}} = R_{\mathrm{b}} + R_{\mathrm{s}} + R_{\mathrm{x}} \tag{2.32}$$

where, R_{b} is the flexural deformation angle of the member given as follows (see Fig.2.19).

$$R_{\mathrm{b}} = \frac{\phi_{\mathrm{y}}\, a}{3} + (\phi_{\mathrm{u}} - \phi_{\mathrm{y}}) l_{\mathrm{p}} \left(1 - \frac{l_{\mathrm{p}}}{2a}\right) \tag{2.33}$$

where, ϕ_y is the member curvature (rad/m) at damage limit, a is the shear span (m) which is half of the clear span, ϕ_u is the hinge curvature (rad/m) at the maximum capacity of the member, and l_p is the hinge length (m).

R_s is the shear deformation angle (rad) of the member at safety limit, and R_x is the deformation angle (rad) depending on the joints of adjacent members and the type of structural system.

The story drift at safety limit shall not exceed 1/75 (or 1/30 for wooden construction) of the story height.

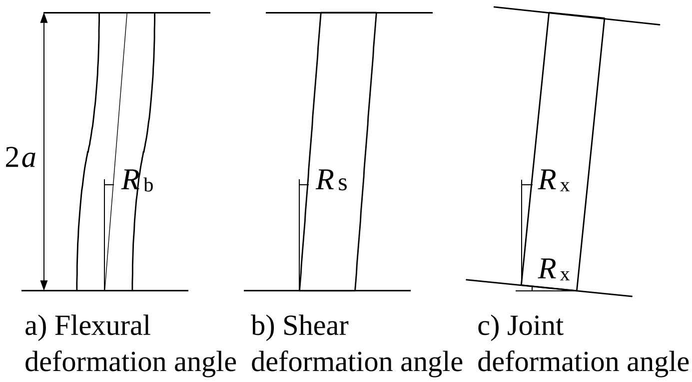
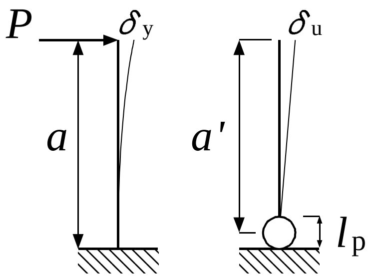


Figure 2.18: Three kinds of deformation angle of a column



$$\delta_y = \frac{Pa^3}{3EI} \qquad \phi_y = \frac{Pa}{EI}$$
$$R_y = \frac{\delta_y}{a} = \frac{Pa^2}{3EI} = \frac{\phi_y a}{3}$$
$$\delta_u = (\phi_u - \phi_y)\, l_p\, (a - l_p/2)$$
$$R_u = \frac{\delta_u}{a} = (\phi_u - \phi_y)\, l_p\, (1 - \frac{l_p}{2a})$$

Figure 2.19: Flexural deformation δ_y and deformation caused by a hinge δ_u

(iv) Period Adjustment Factor (Route $\boxed{4}$)

The period adjustment factor r at damage limit is calculated as follows.

$$r = \sqrt{1 + \left(\frac{T_{sw}}{T_d}\right)^2 + \left(\frac{T_{ro}}{T_d}\right)^2} \tag{2.34}$$

where, T_{sw} (s) is the response period of sway that is calculated as follows.

$$T_{sw} = 2\pi\sqrt{\frac{M_{ud}}{K_h}} \tag{2.35}$$

where, K_h (kN/m) is the horizontal stiffness of the soil.

T_{ro} is the response period of rocking that is calculated as follows.

$$T_{\mathrm{ro}} = 2\pi \sqrt{\frac{M_{\mathrm{ud}}}{K_{\mathrm{r}}} H} \tag{2.36}$$

where, K_{r} (kN m/rad) is the rotational stiffness of the soil, and H (m) is the equivalent height of the building from the bottom of the base.

The period adjustment factor r at safety limit is calculated in the same manner as Eqs.(2.34), (2.35) and (2.36), where T_{d} and M_{ud} are replaced by T_{s} and M_{us}, respectively.

(2) Acceleration Distribution Factor (Route $\boxed{4}$)

(i) Acceleration Distribution Factor at Damage Limit (Route $\boxed{4}$)

The acceleration distribution factor $B_{\mathrm{d}i}$ of the i-th story at damage limit is calculated from the product of p, q and the participation function which corresponds to the response period and the distribution of displacements at damage limit.

For regular buildings or buildings of not more than five stories, $B_{\mathrm{d}i}$ can be calculated as follows.

$$B_{\mathrm{d}i} = p\, q\, r_{\mathrm{m}}\, b_{\mathrm{d}i} \tag{2.37}$$

where, the adjustment factor p is given in Table 2.9 and q is calculated as follows.

$$q = \begin{cases} 0.75 \dfrac{\sum m_i}{M_{\mathrm{ud}}} & \text{for } r_{\mathrm{m}} < 0.75 \\ 1 & \text{for } r_{\mathrm{m}} \geqq 0.75 \end{cases} \tag{2.38}$$

The effective mass ratio r_{m} is given as follows.

Table 2.9: Adjustment factor p

Number of stories	Response period (s) at damage limit	
	$T_{\mathrm{d}} \leqq 0.16$	$0.16 \leqq T_{\mathrm{d}}$
1	$1.00 - (0.20/0.16)T_{\mathrm{s}}$	0.80
2	$1.00 - (0.15/0.16)T_{\mathrm{s}}$	0.85
3	$1.00 - (0.10/0.16)T_{\mathrm{s}}$	0.90
4	$1.00 - (0.05/0.16)T_{\mathrm{s}}$	0.95
5 or more	1.00	1.00

$$r_{\mathrm{m}} = \frac{M_{\mathrm{ud}}}{\sum_{i=1}^{n} m_i} \tag{2.39}$$

where, M_{ud} is given in Eq.(2.27) and $b_{\mathrm{d}i}$ is calculated as follows.

$$b_{\mathrm{d}i} = \begin{cases} 1 + (\sqrt{\alpha_i} - \alpha_i^2)\dfrac{2T}{1+3T}\dfrac{\sum_{i=1}^{n} m_i}{m_n} & \text{if } i = n \\ 1 + (\sqrt{\alpha_i} - \sqrt{\alpha_{i+1}} - \alpha_i^2 + \alpha_{i+1}^2)\dfrac{2T}{1+3T}\dfrac{\sum_{i=1}^{n} m_i}{m_i} & \text{if } i \neq n \end{cases} \tag{2.40}$$

where, α_i is the normalized weight of the i-th story that is given by Eq.(2.18) on p.37 and used in Eq.(2.17) on p.36 and T (s) is the fundamental natural period of the building in Eq.(2.19) on p.37.

(ii) Acceleration Distribution Factor at Safety Limit (Route $\boxed{4}$)

The acceleration distribution factor B_{si} at safety limit is calculated in the same manner as B_{di} at damage limit, replacing T_d, M_{ud} and b_{di} by T_s, M_{us} and b_{si}, respectively.

Comparison of A_i and b_{di}

The normalized weight α_i given by Eq.(2.18) on p.37 becomes as follows.

$$\alpha_i = \frac{\sum_{j=i}^n w_j}{\sum_{j=1}^n w_j} = \frac{\sum_{j=i}^n m_j\, g}{\sum_{j=1}^n m_j\, g} = \frac{\sum_{j=i}^n m_j}{\sum_{j=1}^n m_j} \tag{2.41}$$

The shear of the i-th story normalized by the base shear (normalized shear) is given by $A_i\,\alpha_i$, where A_i is given by Eq.(2.17) on p.36, and it becomes as follows (see Fig.2.20).

$$A_i\,\alpha_i = \left\{1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right)\frac{2T}{1+3T}\right\}\alpha_i = \alpha_i + \left(\sqrt{\alpha_i} - \alpha_i^2\right)\frac{2T}{1+3T} \tag{2.42}$$

The normalized shear of the $(i+1)$-th story becomes as follows.

$$A_{i+1}\,\alpha_{i+1} = \alpha_{i+1} + \left(\sqrt{\alpha_{i+1}} - \alpha_{i+1}^2\right)\frac{2T}{1+3T} \tag{2.43}$$

Subtracting Eq.(2.43) from Eq.(2.42), the normalized seismic force (normalized seismic shear increment) of the i-th story becomes as follows.

$$A_i\,\alpha_i - A_{i+1}\,\alpha_{i+1} = \alpha_1 - \alpha_{i+1} + \left(\sqrt{\alpha_i} - \sqrt{\alpha_{i+1}} + \alpha_i^2 - \alpha_{i+1}^2\right)\frac{2T}{1+3T} \tag{2.44}$$

Dividing the above equation by the normalized weight increment of the i-th story, i.e. $(\alpha_i - \alpha_{i+1})$, we have the normalized seismic coefficient as folows.

$$\frac{A_i\,\alpha_i - A_{i+1}\,\alpha_{i+1}}{\alpha_i - \alpha_{i+1}} = 1 + \left(\sqrt{\alpha_i} - \sqrt{\alpha_{i+1}} + \alpha_i^2 - \alpha_{i+1}^2\right)\frac{2T}{1+3T}\,\frac{1}{\alpha_i - \alpha_{i+1}} \tag{2.45}$$

Substituting the next relationship into the last factor, then

$$\frac{1}{\alpha_i - \alpha_{i+1}} = \frac{\sum_{j=1}^n m_j}{m_i}$$

$$1 + (\sqrt{\alpha_i} - \sqrt{\alpha_{i+1}} - \alpha_i^2 + \alpha_{i+1}^2)\frac{2T}{1+3T}\,\frac{\sum_{i=1}^n m_i}{m_i} = b_{di} \tag{2.46}$$

This is the second equation of Eq.(2.40).

As to the uppermost story, $i = n$ then α_i becomes as follows.

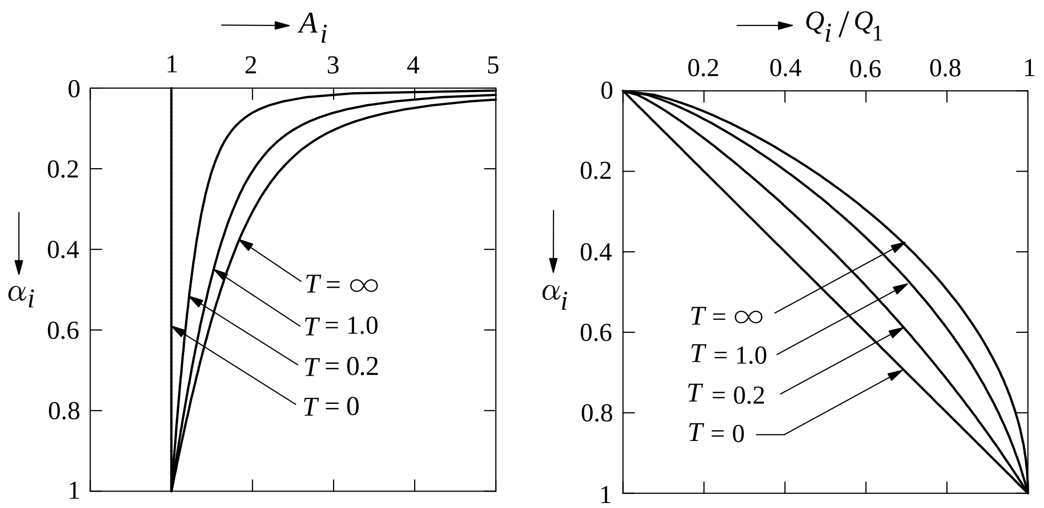
$$\alpha_i = \frac{\sum_{j=i}^{n} m_j}{\sum_{j=1}^{n} m_j} = \frac{m_n}{\sum_{j=1}^{n} m_j} \tag{2.47}$$

Substituting this relationship into the first equation of Eq.(2.40), $b_{\mathrm{d}i}$ become as follows.

$$b_{\mathrm{d}i} = 1 + (\sqrt{\alpha_i} - \alpha_i^2)\frac{2T}{1+3T}\frac{1}{\alpha_i} = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right)\frac{2T}{1+3T} = A_i \tag{2.48}$$

Then $b_{\mathrm{d}i}$ is equal to A_i at the uppermost story. This is because the normalized seismic coefficient is identical to the normalized seismic shear coefficient for the uppermost story.

Therefore, A_i and $b_{\mathrm{d}i}$ give the same distribution of seismic force parameters, although A_i gives the distribution of (normalized) seismic shear coefficient and $b_{\mathrm{d}i}$ gives the distribution of (normalized) seismic coefficient.



a) A_i distribution b) $Q_i/Q_1 = A_i\,\alpha_i$ distribution

Figure 2.20: Normalized shear coefficient A_i and normalized shear $Q_i/Q_1 = A_i\,\alpha_i$

(3) Surface Soil Amplification Factor (Route $\boxed{4}$)

The surface soil amplification factor G_{s} is calculated as either of the procedure 1) or 2).

1) The surface soil amplification factor G_{s} for Soil Profile Type 1 is calculated as follows.

$$G_{\mathrm{s}} = \begin{cases} 1.5 & \text{for } T < 0.576 \\ 0.864/T & \text{for } 0.576 \leqq T < 0.64 \\ 1.35 & \text{for } 0.64 \leqq T \end{cases} \tag{2.49}$$

where, $T(\mathrm{s})$ is the fundamental natural period of the building.

G_s for Soil Profile Type 2 or Type 3 is calculated as follows.

$$G_s = \begin{cases} 1.5 & \text{for} \;\; T < 0.64 \\ 1.5T/0.64 & \text{for} \;\; 0.64 \leqq T < T_u \\ g_v & \text{for} \;\; T_u \leqq T \end{cases} \tag{2.50}$$

where, the transition period T_u is given as follows:

$$T_u = 0.64\left(\frac{g_v}{1.5}\right) \tag{2.51}$$

and the basic amplification factor g_v is given as follows:

$$g_v = \begin{cases} 2.025 & \text{for Soil Profile Type 2} \\ 2.7 & \text{for Soil Profile Type 3} \end{cases} \tag{2.52}$$

Plots of G_s for the three soil profile types are shown in Fig.2.21.

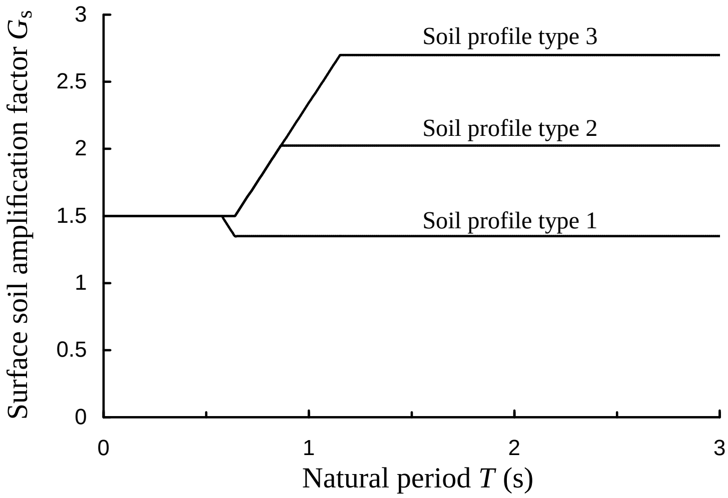


Figure 2.21: Surface soil amplification factor G_s

2) The surface soil amplification factor G_s at safety limit can be calculated using Table 2.10 (see Fig.2.22). The factor can be multiplied by the building-soil interaction factor β.

In Table 2.10, T_1 is the fundamental natural period and T_2 is the second natural period of the surface soil that can be calculated as follows.

$$T_1 = \frac{4\left(\sum H_i\right)^2}{\sum \sqrt{\dfrac{G_i}{\rho_i}} H_i} \qquad T_2 = \frac{T_1}{3} \tag{2.53}$$

where, H_i is the height of the i-th soil layer, ρ_i is the density of the i-th soil layer, and the shear modulus of the i-th soil layer G_i is calculated as follows.

$$G_i = r_{Gi}\, G_{0i} \tag{2.54}$$

Table 2.10: Surface soil amplification factor G_s

T	G_s
$T \leqq 0.8\,T_2$	$G_{s2}\,\dfrac{T}{0.8\,T_2}$
$0.8\,T_2 < T \leqq 0.8\,T_1$	$\dfrac{G_{s1}-G_{s2}}{0.8\,(T_1-T_2)}\,T + G_{s2} - 0.8\,\dfrac{G_{s1}-G_{s2}}{0.8\,(T_1-T_2)}\,T_2$
$0.8\,T_1 < T \leqq 1.2\,T_1$	G_{s1}
$1.2\,T_1 < T$	$\dfrac{G_{s1}-1}{\dfrac{1}{1.2\,T_1}-0.1}\,\dfrac{1}{T} + G_{s1} - \dfrac{G_{s1}-1}{\dfrac{1}{1.2\,T_1}-0.1}\,\dfrac{1}{1.2\,T_1}$

T: Response period of the building at safety limit
T_1, T_2: First and second predominant periods of surface soil
G_{s1}, G_{s2}: First and second amplification factor of surface soil

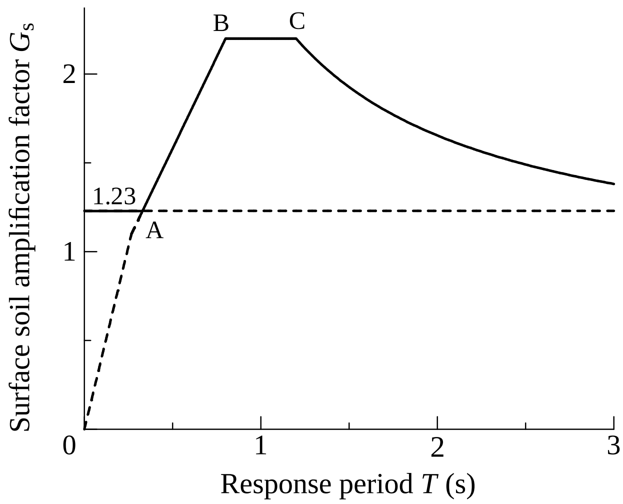


Figure 2.22: Surface soil amplification factor G_s (Route $\boxed{4}$)
(G_{s1} and G_{s2} are calculated from Eqs.(2.56) and (2.57), assuming $\zeta = 0.15$, $\alpha = 0.2$ and $T_1 = 1$ (s))

where, r_{Gi} is the shear modulus reduction factor of of the i-th soil layer, and the basic shear modulus of the i-th soil layer G_{0i} is calculated as follows.

$$G_{0i} = \rho_i\, V_{si}^2 \tag{2.55}$$

where, V_{si} is the shear wave velocity of the i-th soil layer.

G_{s1} and G_{s2} are calculated as follows.

$$G_{s1} = \frac{1}{1.57\,\zeta + \alpha} \geqq 1.2 \tag{2.56}$$

$$G_{s2} = \frac{1}{4.71\,\zeta + \alpha} \tag{2.57}$$

where, α is the wave impedance ratio that is calculated as follows.

$$\alpha = \frac{\sum \sqrt{\frac{G_i}{\rho_i}} H_i \sum \rho_i H_i}{(\sum H_i)^2} \frac{1}{\rho_B V_B} \tag{2.58}$$

where, ρ_B is the density of the engineering bedrock, and V_B is the shear wave velocity of the engineering bedrock.

The damping ratio of the surface soil is calculated as follows.

$$\zeta = 0.8 \frac{\sum \zeta_i E_i}{\sum E_i} \geqq 0.05 \tag{2.59}$$

where, ζ_i is the damping ratio of the ***i***-th soil layer, and E_i is the maximum elastic strain energy of the ***i***-th soil layer that is calculated as follows.

$$E_i = \frac{G_i}{2H_i}(u_i - u_{i-1})^2 \tag{2.60}$$

where, u_i is the displacement of the ***i***-th soil layer relative to the engineering bedrock.

The building-soil interaction factor β is calculated as follows.

$$\beta = \frac{K_{hb}\left\{1 - (1 - \frac{1}{G_s})\frac{D_e}{\sum H_i}\right\} + K_{he}}{K_{hb} + K_{he}} \geqq 0.75 \tag{2.61}$$

where, K_{hb} is the horizontal stiffness of the soil at the bottom of the basement of the building, G_s is calculated using the equations in Table 2.10, D_e is the depth to the bottom of the basement from the ground surface, and K_{he} is the horizontal stiffness of the soil at the wall of the basement.

For reference the shear modulus reduction factor of soil and the damping ratio of soil in the previous notification are shown in Fig.2.23.

(4) Acceleration Reduction Factor (Route $\boxed{4}$)

The acceleration reduction factor F_ζ is calculated using Eq.(2.62) (see the solid curve in Fig.2.24), or it can be calculated by considering damping effect of the building or its members against earthquake response.

$$F_\zeta = \frac{1.5}{1 + 10\,\zeta} \tag{2.62}$$

where, ζ is the damping ratio of the building.

The damping ratio ζ is calculated by one of the following procedures **1)**, **2)** or **3)**.

1) The damping ratio ζ of a building is calculated from the damping ratio ${}_m\zeta_{ei}$ of each member at safety limit as follows.

$$\zeta = \frac{\sum_{i=1}^{N} {}_m\zeta_{ei}\, {}_mE_i}{\sum_{i=1}^{N} {}_mE_i} + 0.05 \tag{2.63}$$

where, N is the number of structural members of the building, ${}_mE_i$ is the product of the deformation and the capacity at safety limit of the member divided by 2.

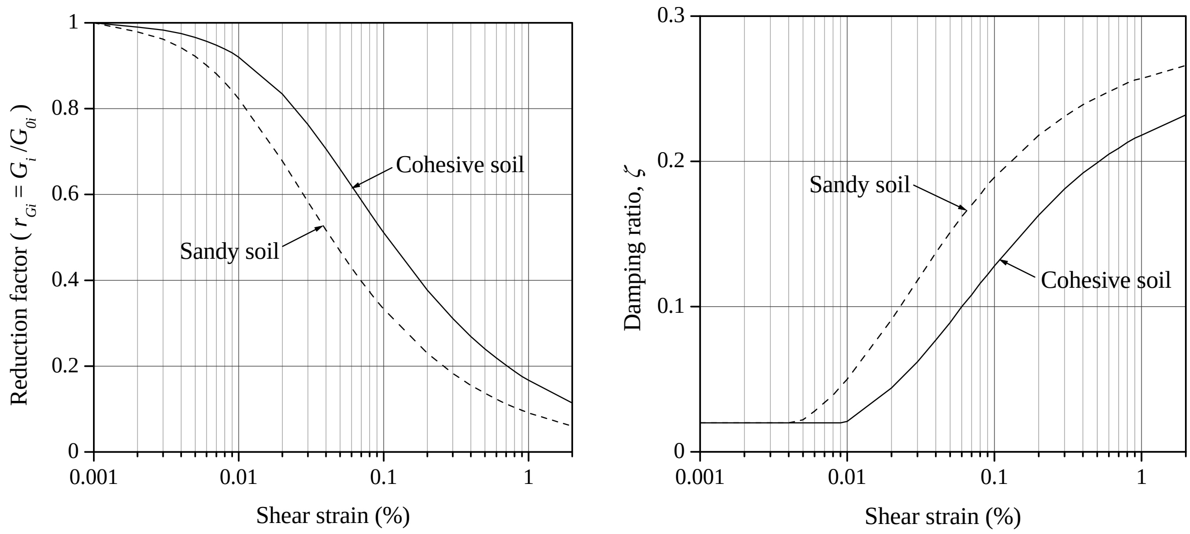
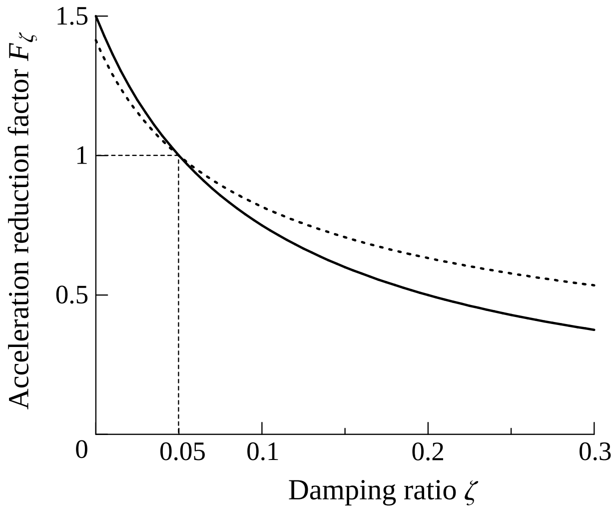


Figure 2.23: Shear modulus reduction factor and damping of soil (in the previous notification)



(The solid curve indicates Eq.(2.62), and the dotted curve is Eq.(3.18), the same factor of Eurocode 8.)

Figure 2.24: Acceleration reduction factor F_ζ

i) The damping ratio ${}_{\mathrm{m}}\zeta_{\mathrm{e}i}$ of wood, steel or reinforced concrete member is calculated as follows.

$$ {}_{\mathrm{m}}\zeta_{\mathrm{e}i} = \gamma_1 \left(1 - \frac{1}{\sqrt{{}_{\mathrm{m}}D_{\mathrm{f}i}}}\right) \tag{2.64} $$

where,

$$ \gamma_1 = \begin{cases} 0.25 & \text{for members rigidly connected to adjacent members} \\ 0.20 & \text{for members other than the above and compression braces} \end{cases} \tag{2.65} $$

${}_{\mathrm{m}}D_{\mathrm{f}i}$ is a ductility parameter of the member that is calculated as follows.

$$ {}_{\mathrm{m}}D_{\mathrm{f}i} = \frac{{}_{\mathrm{m}}\delta_{\mathrm{s}i}}{{}_{\mathrm{m}}\delta_{\mathrm{d}i}} \geqq 1 \tag{2.66} $$

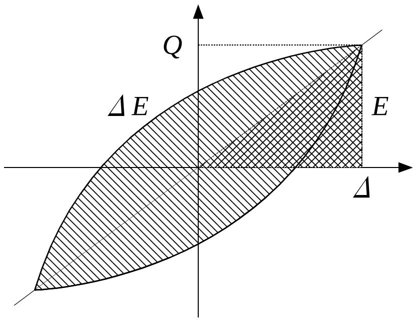


Figure 2.25: Strain energy E and absorbed energy ΔE

where, ${}_{\mathrm{m}}\delta_{\mathrm{s}i}$ is the deformation of the member at safety limit, and ${}_{\mathrm{m}}\delta_{\mathrm{d}i}$ is the deformation of the member at damage limit.

ii) The damping ratio ${}_{\mathrm{m}}\zeta_{\mathrm{e}i}$ of the member other than wood, steel or reinforced concrete is calculated as the equivalent viscous damping ratio of the member at safety limit of Eq.(2.67) multiplied by 0.8. The damping ratio shall not exceed the value obtained from Eq.(2.64) for $\gamma_1 = 0.25$.

$$ {}_{\mathrm{m}}\zeta_{\mathrm{e}i} = \frac{1}{4\pi}\frac{\Delta E_i}{{}_{\mathrm{m}}E_i} \tag{2.67}$$

where, ΔE_i is the energy absorbed by the member, calculated as the enclosed area of the hysteresis curve at safety limit, and ${}_{\mathrm{m}}E_i$ is the product of the deformation and the capacity at safety limit of the member divided by 2 (see Fig.2.25).

2) In case all γ_1's of ${}_{\mathrm{m}}D_{\mathrm{f}i}$ that exceeds 1 are equal, the damping ratio ζ of the building can be calculated as follows.

$$\zeta = \gamma_1\left(1 - \frac{1}{\sqrt{D_{\mathrm{f}}}}\right) + 0.05 \tag{2.68}$$

where, D_{f} is a ductility parameter of the building that is calculated as follows (see Fig.2.26).

$$D_{\mathrm{f}} = \frac{\Delta_{\mathrm{s}}\, Q_{\mathrm{d}}}{\Delta_{\mathrm{d}}\, Q_{\mathrm{s}}} \geqq 1 \tag{2.69}$$

where, Δ_{d} is the displacement at damage limit, Δ_{s} is the displacement at safety limit, Q_{d} is the lateral capacity at damage limit and Q_{s} is the lateral capacity at safety limit.

3) In case the relationships between the lateral force and the lateral displacement of the bulding are given, the damping ratio of the building can be calculated as follows.

$$\zeta = \gamma_1\left(1 - \frac{1}{\sqrt{D_{\mathrm{f}}}}\right) + 0.05 \tag{2.70}$$

where, γ_1 is the same as in procedure **1)**, and D_{f} is the ductility factor of the bulding that is given as follows.

$$D_{\mathrm{f}} = \frac{\Delta_{\mathrm{s}}}{\Delta_{\mathrm{y}}} \geqq 1 \tag{2.71}$$

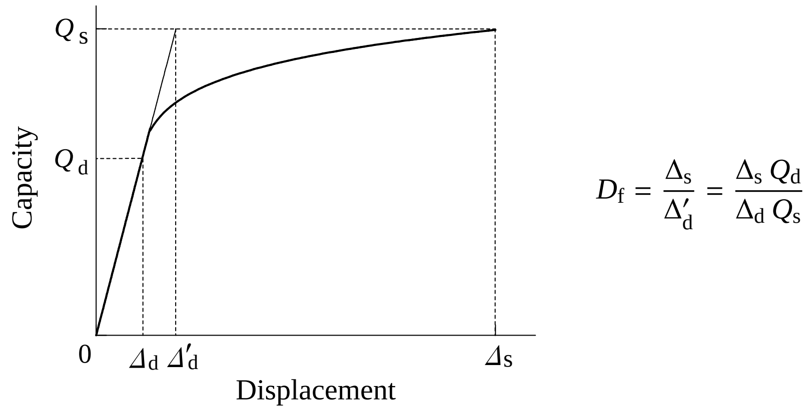


Figure 2.26: Ductility parameter D_f, damage limit (Δ_d, Q_d) and safety limit (Δ_s, Q_s)

where, Δ_s(m) is the equivalent displacement of the buliding at safety limit, and Δ_y(m) is the yield displacement of the bi-linear model for the relationship between the lateral capacity and the lateral displacement of the bulding (see Fig.2.27).

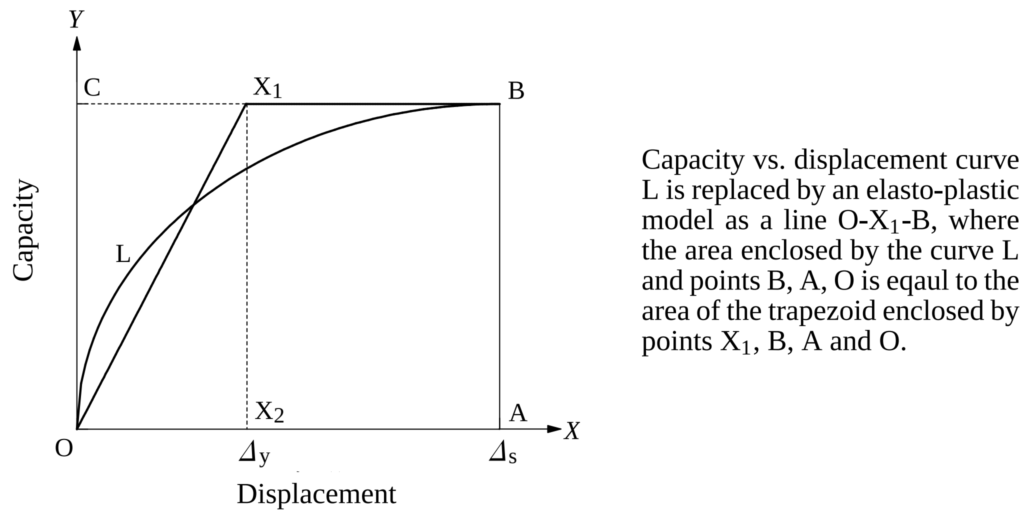


Figure 2.27: Lateral capacity and displacement

4) In the case where the soil conditions are investigated by ground survey, the equivalent viscous damping ratio ζ of the building can be calculated as follows.

$$\zeta = \frac{1}{r^3}\left\{\zeta_{sw}\left(\frac{T_{sw}}{T_d}\right)^3 + \zeta_{ro}\left(\frac{T_{ro}}{T_d}\right)^3 + \zeta_b\right\} \tag{2.72}$$

where, r is the period adjustment factor, $\zeta_{sw} \leqq 0.3$ is the viscous damping ratio for sway of the surface soil corresponding to its shear strain, $\zeta_{ro} \leqq 0.15$ is the viscous damping ratio for rocking of the surface soil corresponding to its shear strain, and ζ_b is the equivalent viscous damping ratio calculated in either previous procedure **1)** or **2)**.

2.6 Notations

Each notation is defined where it appears for the first time. This is the summary of the main notations used in this chapter (Chap.2).

Roman notations

A_c	cross-sectional area of column	柱の水平断面積
A_i	lateral shear distribution factor	層せん断力分布係数
A_w	horizontal cross-sectional area of shear wall	耐震壁の水平断面積
a	shear span	せん断スパン
$B_{\mathrm{d}i}$	acceleration distribution factor at damage limit	損傷限界の加速度分布係数
$B_{\mathrm{s}i}$	acceleration distribution factor at safety limit	安全限界の加速度分布係数
$b_{\mathrm{d}i}$	parameter for calculating $B_{\mathrm{d}i}$	$B_{\mathrm{d}i}$ を計算するときに用いる値
C_0	standard shear coefficient	標準せん断力係数
C_i	lateral seismic shear coefficient of the i-th story	i 階の地震層せん断力係数
D_f	ductility parameter of the building	建築物の塑性率を表すパラメータ
D_e	depth to the bottom of the basement from the ground surface	地表面から地下室底面までの深さ
${}_\mathrm{m}D_{\mathrm{f}i}$	ductility parameter of the member	部材の塑性率を表すパラメータ
D_s	structural characteristic factor	構造特性係数
E_i	maximum elastic strain energy of the i-th soil layer	i 番目の地層の最大弾性ひずみエネルギー
${}_mE_i$	product of the deformation and the capacity at safety limit of the member divided by 2	部材の安全限界時の変形と耐力の積の 1/2
ΔE_i	energy absorbed by the member	部材の吸収エネルギー
e	eccentricity	偏心距離
F	standard strength of steel	鋼材の基準強度
F_c	concrete strength	コンクリート強度
F_e	shape factor as a function of stiffness eccentricity ratio	偏心率による形状係数
$F_{\mathrm{e}i}$	shape factor of the i-th story as a function of stiffness eccentricity ratio	i 階の偏心率による形状係数
F_es	shape factor	形状係数
F_ζ	acceleration reduction factor	加速度の低減率
F_s	shape factor as a function of lateral stiffness ratio	剛性率による形状係数
G_{0i}	basic shear modulus of the i-th soil layer	i 番目の地層の基本せん断剛性
G_i	shear modulus of the i-th soil layer	i 番目の地層のせん断剛性
G_s	surface soil amplification factor	表層地盤の増幅係数
$G_{\mathrm{s}1}$	surface soil amplification factor for the first predominant period	表層地盤の 1 次卓越周期に対する増幅係数
$G_{\mathrm{s}2}$	surface soil amplification factor for the second predominant period	表層地盤の 2 次卓越周期に対する増幅係数
g	acceleration due to gravity	重力加速度
g_v	basic soil amplification factor	地盤の基本増幅係数

H	depth of the basement	地下の深さ
H	equivalent height of the building from the bottom of the base	建築物の基礎底面からの代表高さ
H_i	height of the *i*-th soil layer	*i* 番目の地層の層厚
h	height of the building	建築物の高さ
K_{h}	horizontal stiffness of the soil	水平地盤ばね定数
K_{hb}	horizontal stiffness of the soil at the bottom of the basement	地下底面の水平地盤ばね定数
K_{he}	horizontal stiffness of the soil at the wall of the basement	地下側壁の水平地盤ばね定数
K_{r}	rotational stiffness of the soil	回転地盤ばね定数
k	design seismic coefficient of the basement	地下の設計用震度
l_{p}	hinge length	ヒンジ領域の長さ
M_{D}	design bending moment	設計用曲げモーメント
M_{ud}	effective mass of the building at damage limit	建築物の損傷限界有効質量
M_{us}	effective mass of the building at safety limit	建築物の安全限界有効質量
M_{W}	bending moment of the shear wall when the building come to its global collapse	全体崩壊形に達する場合に耐力壁に作用する曲げモーメント
m_i	mass of the *i*-th story	*i* 階の質量
N	number of structural members of a building	建築物の部材の総数
n	number of stories of the building above the ground level	建築物の地上階数
n	factor to be used calculating design shear	設計用せん断力を求める際に用いる係数
n_1	factor to be used calculating design shear of the shear wall	耐力壁の設計用せん断力を求める際に用いる係数
n_2	factor to be used calculating design bending moment of the shear wall	耐力壁の設計用曲げモーメントを求める際に用いる係数
$P_{\mathrm{d}i}$	lateral seismic force of the *i*-th story at damage limit	*i* 階の損傷限界水平方向地震力
$P^*_{\mathrm{d}i}$	lateral force of the *i*-th story at damage limit	*i* 階の損傷限界水平力
$P_{\mathrm{s}i}$	lateral seismic force of the *i*-th story at safety limit	*i* 階の安全限界水平方向地震力
$P^*_{\mathrm{s}i}$	lateral force of the *i*-th story at safety limit	*i* 階の安全限界水平力
p	adjustment factor to calculate $B_{\mathrm{d}i}$	$B_{\mathrm{d}i}$ を計算するための調整係数
Q	lateral seismic shear	地震層せん断力
Q_0	shear of the member caused by permanent load assuming both ends are simply supported	部材の両端が単純支持の場合の常時荷重によって生ずるせん断力
Q_{D}	design shear	設計用せん断力
Q_{d}	lateral capacity of the building at damage limit	建築物の損傷限界耐力
$Q_{\mathrm{d}i}$	lateral capacity of the *i*-th story at damage limit	*i* 階の損傷限界耐力
Q_{B}	lateral seismic shear of the basement	地下の地震層せん断力
Q_{b}	increased lateral seismic shear	割増した地震層せん断力
Q_{E}	shear caused by moderate earthquake motions	中地震動によって生ずるせん断力
Q_i	lateral seismic shear of the *i*-th story	*i* 階の地震層せん断力
Q_{L}	shear caused by permanent load	常時荷重によって生ずるせん断力
Q_{p}	portion of the lateral seismic shear of the first story that will extend to the basement	地下に伝達される 1 階の地震層せん断力
Q_{s}	lateral capacity of the building at safety limit	建築物の安全限界耐力

Q_u	shear of column or beam when all adjoining beams reach their maximum flexural capacities	すべての境界梁の材端が最大曲げモーメントに達した時の柱・梁に生じうるせん断力
Q_{ud}	lateral seismic shear for severe earthquake motions	大地震動時の地震層せん断力
Q_{ui}	ultimate lateral capacity of the *i*-th story	*i* 階の保有水平耐力
Q_{un}	specified lateral shear	必要保有水平耐力
Q_W	shear of the shear wall when the building reach its global collapse mechanism	全体崩壊形に達する場合に耐力壁に作用するせん断力
Q_y	shear of the member assuming both ends yield in bending	両端が曲げ降伏となる部材に生ずるせん断力
q	factor to calculate B_{di} depending on the effective mass ratio	B_{di} を計算するための有効質量比による係数
q_{di}	base shear coefficient calculated from the *i*-th story lateral capacity at damage limit	*i* 階の損傷限界の 1 階の換算層せん断力係数
q_{si}	base shear coefficient calculated from the *i*-th story lateral capacity at safety limit	*i* 階の安全限界の 1 階の換算層せん断力係数
R_b	flexural deformation angle at safety limit	曲げに対する安全限界変形角
R_e	stiffness eccentricity ratio	偏心率
R_s	lateral stiffness ratio	剛性率
R_s	shear deformation angle at safety limit	安全限界せん断変形角
R_t	design spectral factor	振動特性係数
R_u	deformation angle of the member at safety limit	部材の安全限界変形角
R_x	deformation angle depending on the joints of adjacent members and the types of structural systems	周辺部材との接合や構造形式による変形角
r	period adjustment factor	周期調整係数
r_s	lateral stiffness	水平剛性
$\overline{r_s}$	mean lateral stiffness	平均水平剛性
r_e	elastic radius	弾力半径
r_{Gi}	shear modulus reduction factor of of the *i*-th soil layer	*i* 番目の地層のせん断剛性低減係数
r_m	effective mass ratio	有効質量比
S_{Ad}	acceleration response at damage limit on the engineering bedrock	工学的基盤における損傷限界の加速度応答
S_{Ae}	acceleration response for extremely rare earthquake on the engineering bedrock	工学的基盤における極めて希に起こる地震の加速度応答
S_{Ar}	acceleration response for rare earthquake on the engineering bedrock	工学的基盤における希に起こる地震の加速度応答
S_{As}	acceleration response at safety limit on the engineering bedrock	工学的基盤における安全限界の加速度応答
T	(fundamental) natural period (of the building)	（建築物の 1 次）固有周期
T	response period of the building at damage limit or safety limit	損傷限界または安全限界に対する建築物の応答周期
T_1	fundamental natural period (of the surface soil)	（表層地盤の）1 次固有周期
T_2	second natural period (of the surface soil)	（表層地盤の）2 次固有周期
T_c	critical period of the soil	地盤周期
T_d	response period of the building at damage limit	建築物の損傷限界応答周期
T_{ro}	response period of rocking	ロッキング応答周期
T_s	response period of the building at safety limit	建築物の安全限界応答周期

T_{sw}	response period of sway	スウェイ応答周期
T_{u}	transition period of surface soil amplification factor	表層地盤の増幅係数の移行周期
u_i	displacement of the *i*-th soil layer relative to the engineering bedrock	*i* 番目の地層の工学的基盤からの相対変位
V_{B}	shear wave velocity of the engineering bedrock	工学的基盤のせん断波速度
$V_{\mathrm{s}i}$	shear wave velocity of the *i*-th soil layer	*i* 番目の地層のせん断波速度
W	weight of the building above the ground level	地上部分の建築物の全重量
W_{B}	weight of the basement	地下の重量
W_i	weight of the building above the *i*-th story	*i* 階より上の建築物の重量
w_i	weight of the *i*-th story of the building	建築物の *i* 階の重量
Z	seismic hazard zoning factor	地震地域係数

Greek notations

α	wave impedance ratio	波動のインピーダンス比
α_i	normalized weight	基準化重量
α	adjusting factor depending on the concrete strength	コンクリート強度による調整係数
β	ratio of the lateral shear of braces to the total lateral seismic shear of the story	筋かいの水平力分担率
β	building-soil interaction factor	建築物と地盤の相互作用係数
γ_1	factor to be used calculating damping ratio of the member	部材の減衰定数を求める際に用いる係数
Δ_{d}	equivalent displacement of the building at damage limit	建築物の損傷限界代表変位
Δ_{s}	equivalent displacement of the building at safety limit	建築物の安全限界代表変位
$\delta_{\mathrm{d}i}$	displacement of the *i*-th story at damage limit	損傷限界時の *i* 階の変位
$\delta_{\mathrm{s}i}$	displacement of the *i*-th story at safety limit	安全限界時の *i* 階の変位
${}_{\mathrm{m}}\delta_{\mathrm{d}i}$	deformation of the member at damage limit	部材の損傷限界変形
${}_{\mathrm{m}}\delta_{\mathrm{s}i}$	deformation of the member at safety limit	部材の安全限界変形
ζ	damping ratio	減衰定数
ζ_{b}	damping ratio of the building at damage limit	損傷限界の建築物の減衰定数
ζ_i	damping ratio of the *i*-th soil layer	*i* 番目の地層の減衰定数
ζ_{sw}	damping ratio for sway	スゥェイの減衰定数
ζ_{ro}	damping ratio for rocking	ロッキングの減衰定数
${}_{\mathrm{m}}\zeta_{\mathrm{e}i}$	damping ratio of the member at safety limit	部材の安全限界減衰定数
λ	ratio of the total height of stories of wooden or steel construction to the height of the building	木造または鉄骨造の階の高さと建築物の高さの比
ρ_B	density of the engineering bedrock	工学的基盤の密度
ρ_i	density of the *i*-th soil layer	*i* 番目の地層の密度
ϕ_{u}	hinge curvature at the maximum capacity	最大耐力時のヒンジ領域での曲率
ϕ_{y}	member curvature at damage limit	損傷限界時の部材の曲率

Chapter 3

Seismic Codes in the World

3.1 ISO 3010 - Seismic Actions on Structures

1) Introduction

The International Organization for Standardization (ISO) is a worldwide federation of national standards bodies. The work of preparing International Standards is normally carried out through ISO technical committees (TC's). Each TC has several sub-committees (SC's) and each SC usually has several working groups (WG's).

ISO/TC98 is one of the TC's which deals with "Bases for design of structures". The aim of TC98 is to create a coherent design system of International Standards in the field of building and civil engineering works. The system forms a basis for regional and national standard bodies that prepare their standards for particular types of structures and structural materials. TC98 has three main tasks that are shared among three SC's; (1) terminology and symbols, (2) reliability of structures, and (3) loads, forces and other actions on structures[2].

The first edition of International Standard "ISO 3010 Bases for design of structures - Seismic actions on structures"[3] was published in 1988 and the second edition[4] in 2001 through the activity of the working group in ISO/TC98. ISO 3010 includes principles for the determination of seismic actions on structures and seismic design. Since it does not give any specific values for factors to determine seismic loadings, it is not possible to design a structure only according to ISO 3010. Its annexes, however, give useful information to determine the values for those factors. ISO 3010 includes almost all items and factors to be considered. Therefore it is a useful document for establishing a new code or revising an old one.

2) Outline of Normative Text of ISO 3010

(1) Scope

ISO 3010[4] specifies principles of evaluating seismic actions for the seismic design of buildings, towers, chimneys and similar structures. Some of the principles can be referred to for seismic design of structures such as bridges, dams, harbour installations, tunnels, fuel storage tanks, chemical plants and conventional power plants.

(2) Normative reference

The following normative document contains provision which constitute provisions of this standars.

ISO 2394, General principles on reliability for structures[5]

(3) Terms and definitions

14 terms are shown with their definitions.

(4) Symbols and abbreviated terms

19 symbols and abbreviated terms are shown.

(5) Bases of seismic design

The basic philosophy of seismic design of structures is, in the event of earthquakes,

- to prevent human casualties,
- to ensure continuity of vital services, and
- to minimize damage to property.

It is recognized that to give complete protection against all earthquakes is not economically feasible for most types of structures. This standard states the following basic principles.

a) The structure should not collapse nor experience other similar forms of structural failure due to severe earthquake ground motions that could occur at the site (ultimate limit state: ULS).
b) The structure should withstand moderate earthquake ground motions which may be expected to occur at the site during the service life of the structure with damage within accepted limits (serviceability limit state: SLS).

(6) Priciples of seismic design

a) Construction site
Characteristics of construction sites under seismic actions should be evaluated, taking into account microzonation criteria.

b) Structural configuration
It is recommended that structures have simple forms in both plan and elevation.

(Plan irregularities) Structural elements to resist horizontal seismic actions should be arranged such that torsional effects become as small as possible.

(Vertical irregularities) Changes in mass, stiffness and capacity along the height of the structure should be minimized to avoid damage concentration.

c) Influence of non-structural elements
The building, including non-structural elements as well as structural elements, should be clearly defined as a load-resisting system which can be analysed.

d) Strength and ductility

The structural system and its structural elements (both members and connections) should have both adequate strength and ductility for applied seismic actions.

e) Deformation of the structure

The deformation of the structure under seismic actions should be limited, neither causing malfunction of the structure for moderate earthquake ground motions, nor causing collapse or other similar forms of structural failure for severe earthquake ground motions.

f) Response control systems

Response control sytems for structures, e.g. seismic isolation, can be used to ensure continuous use of the structure for moderate earthquake ground motions and to prevent collapse during severe earthquake ground motions.

g) Foundations

The type of foundation should be selected carefully in accordance with the type of structure and local soil conditions, e.g. soil profile, subsurface irregularitiy, groundwater level.

(7) Principles of evaluating seismic actions

a) Variable and accidental actions

Seismic actions shall be taken either as variable actions or accidental actions.

Structures should be verified against design values of seismic actions for ULS and SLS. The verifivcation for the SLSs may be omitted provided that it is satisfied through the verification for ULSs.

Accidental seismic actions can be considered for structures in regions where seismic activity is low to ensure structural integrity.

b) Dynamic and equivalent static analyses

(Dynamic analysis) A dynamic analysis is highly recommended for specific structures such as slender high-rise buildings and structures with irregularities of geometry, mass distribution or stiffness distribution.

(Equivalent static analysis) Ordinary and regular structures may be designed by the equivalent static method using conventional linear elastic analysis.

c) Criteria for determination of seismic actions

(Seismicity of the region) The seismicity of the region where a structure is to be constructed is usually indicated by a seismic zoning map, which may be based on either the seismic history or on seismotechtonic data of the region, or on a combination of historical and seismotechtonic data.

(Soil conditions) Dynamic properties of the supporting soil layers of the structure should be investigated and considered.

(Dynamic properties of the structure) Dynamic properties, such as periods and modes of vibration and damping properties, should be considered for overall soil structure system.

(Importance of the structures in relation to its use) A higher level of reliability is required for buildings where large numbers of people assemble, or structures which are essential for public well-being during and after the earthquakes.

(Spacial variation of earthquake ground motion) Usually the relative motion between different points of the ground may be disregarded. However, in case of long-span or widely spread structures, this effect and the effect of a travelling wave should be taken into account.

(8) Evaluation of seismic actions by equivalent static analysis

a) Equivalent seismic loadings

In the seismic analysis of structures based on a method using equivalent static loadings, the variable seismic actions for ULS and SLS may be evaluated as follows.

(ULS) The design lateral seismic force of the i-th level of a structure for ULS, $F_{\mathrm{E,u},i}$, may be determined by

$$F_{\mathrm{E,u},i} = \gamma_{\mathrm{E,u}}\, k_{\mathrm{Z}}\, k_{\mathrm{E,u}}\, k_{\mathrm{D}}\, k_{\mathrm{R}}\, k_{\mathrm{F},i} \sum_{j=1}^{n} F_{\mathrm{G},j} \tag{3.1}$$

or the design lateral seismic shear force for ULS, $V_{\mathrm{E,u},i}$, may be used instead of the above seismic force:

$$V_{\mathrm{E,u},i} = \gamma_{\mathrm{E,u}}\, k_{\mathrm{Z}}\, k_{\mathrm{E,u}}\, k_{\mathrm{D}}\, k_{\mathrm{R}}\, k_{\mathrm{V},i} \sum_{j=i}^{n} F_{\mathrm{G},j} \tag{3.2}$$

where, $\gamma_{\mathrm{E,u}}$ is the load factor as related to reliability of the structure for ULS; k_{Z} is the seismic hazard zoning factor to be specified in the national code or other national documents; $k_{\mathrm{E,u}}$ is the representative value of earthquake ground motion intensity for ULS to be specified in national code or other national documents by considering seismicity; k_{D} is the structural factor to be specified for various structural systems according to their ductility, acceptable deformation, restoring force characteristics and overstrength; k_{R} is the ordinate of the normalized design response spectrum, as a function of the fundamental natural period of the structure considering the effect of soil conditions and damping of the structure; $k_{\mathrm{F},i}$ is the seismic force distribution factor of the i-th level to distribute the seismic shear force of the base to each level, which characterizes the distribution of seismic forces in elevation, where $k_{\mathrm{F},i}$ satisfies the condition $\sum k_{\mathrm{F},i} = 1$; $k_{\mathrm{V},i}$ is the seismic shear distribution factor of the i-th level which is the ratio of the seismic shear factor of the i-th level to the seismic shear factor of the base, and characterizes the distribution of seismic shear forces in elevation, where $k_{\mathrm{V},i} = 1$ at the base and usually becomes largest at the top; $F_{\mathrm{G},j}$ is the gravity load at the j-th level of the structure; and n is the number of levels above the base.

(SLS) The design lateral seismic force of the i-th level of a structure for SLS, $F_{\mathrm{E,s},i}$, may be determined by

$$F_{\mathrm{E,s},i} = \gamma_{\mathrm{E,s}}\, k_{\mathrm{Z}}\, k_{\mathrm{E,s}}\, k_{\mathrm{R}}\, k_{\mathrm{F},i} \sum_{j=1}^{n} F_{\mathrm{G},j} \tag{3.3}$$

or the design lateral seismic shear force of the i-th level for SLS, $V_{\mathrm{E,s},i}$, can be used instead of the above seismic force:

$$V_{\mathrm{E,s},i} = \gamma_{\mathrm{E,s}}\, k_{\mathrm{Z}}\, k_{\mathrm{E,s}}\, k_{\mathrm{R}}\, k_{\mathrm{V},i} \sum_{j=i}^{n} F_{\mathrm{G},j} \tag{3.4}$$

where, $\gamma_{\mathrm{E,s}}$ is the load factor as related to reliability of the structure for SLS; $k_{\mathrm{E,s}}$ is the representative value of earthquake ground motion intensity for SLS to be specified in national codes or other national documents by considering the seismicity.

b) Seismic action components and torsion

The two horizontal and vertical components of the earthquake ground motion and their spatial variation, leading to torsional excitation of structures, should be considered.

c) Seismic actions on parts of structures

When the seismic actions for parts of the structure are evaluated by equivalent static analyses, appropriate factors for seismic forces or shear forces should be used taking into account higher mode effects of the structure including the parts.

(9) Evaluation of seismic actions by dynamic analysis

a) General

When performing a dynamic analysis, it is important to consider a proper model and appropriate earthquake ground motions or response spectra.

b) Dynamic analysis procedures

The usual dynamic analysis procedures may be classified as the response spectrum analysis for linear or equivalent linear systems, or the time history analysis for linear or non-linear systems.

c) Response spectrum analysis

A site-specific design spectrum shall be established in the response spectrum analysis. The spectrum shoud be based on the proper damping ratio. Due consideration shoud be given to the amount of expected post-elastic deformation and associated restoring characteristics.

The maximum dynamic response is usually obtained by the superposition method of SRSS (square root of the sum of squares), taking into account the predominant vibration modes into consideration.

The SRSS method does not always lead to conservative values, particularly when two or more natural modes are closely spaced. For this condition, the CQC (complete quadratic combination) method is recommended.

d) Earthquake ground motions for time history analysis

Time history analysis may require several earthquake ground motion records to ensure adequate coverage of expected seismic events. Simulated earthquake ground motions may be used as an alternative.

Appropriate earthquake ground motions should be determined for each limit state, taking into account the seismicity, local soil conditions, return period of historical earthquakes, distance to active faults, errors in the prediction and design service life of the structure.

(Recorded earthquake ground motions) When recorded earthquake ground motions are considered, the following records may be referred to: strong earthquake ground motions recorded at or near the site, or strong earthquake ground motions recorded at other sites with similar geological, topographic and sesimotechtonic characteristics. Usually these earthquake ground motion records have to be scaled according to the corresponding limit state and seismicity of the site.

(Simulated ground motions) Since it is impossible to predict exactly the earthquake ground motions at site in the future, it may be appropriate to use simulated earthquake ground motions as design inputs.

e) Model of the structure

When setting up a model of the structure, it should represent the dynamic properties of the real structure, such as the natural periods and modes of vibration, damping properties and restoring force characteristics, taking into account material and structural ductility.

Consideration should be given to coupling effects of the structure with its foundation and supporting ground, damping in fundamental and higher modes of vibration, restoring force characteristics of the structural elements, effects of non-structural elements, torsional effects, effects of axial deformation of columns and other vertical elements, effects of irregular distribution of lateral stiffness in elevation, and effects of floor diaphram stiffness.

f) Evaluation of analytical results

When dynamic analysis is carried out, the evaluation of seismic actions may be possible solely based on the results of dynamic analysis. However, the evaluation of seismic actions by equivalent static analysis also gives useful information.

(10) Estimation of paraseismic influences

This standard may be used as an introductory approach for paraseismic influences whose characteristics are similar to natural earthquakes, e.g. underground explosion, traffic vibration, pile driving and other human activities.

3) Outline of Informative Annexes of ISO 3010

(1) Load factors as related to the reliability of the structure, seismic hazard zoning factor and representative values of earthquake ground motion intensity

a) Load factors as related to reliability of the structure

$\gamma_{E,u}$ and $\gamma_{E,s}$ are the load factors for ULS and SLS. They are partial factors for action according to the partial factor format in ISO 2394.

$\gamma_{E,u}$ and $\gamma_{E,s}$ are, for example, listed in Tables 3.1 and 3.2 for a region of relatively high seismic hazard, along with the representative values of earthquake ground motion intensity $k_{E,u}$ and $k_{E,s}$. An example using the unity load factor for a normal degree of importance is shown in Table 3.1, while a common representative value k_E is used in Table 3.2.

b) Seismic hazard zoning factor, k_Z

The seismic hazard zoning factor, k_Z, reflects the relative seismic hazard of the region.

c) Representative values of earthquake ground motion intensity

The representative values $k_{E,u}$ and $k_{E,s}$ are usually described in terms of horizontal peak ground acceleration as a ratio to the acceleration due to gravity.

Currently, $k_{E,u}$ is approximately 0.4 at a region with the highest seismic hazard in the world for a return period of approximately 500 years.

Table 3.1: Example 1 for load factors and representative values

Limit state	Degree of importance	Load factor $\gamma_{E,u}$, $\gamma_{E,s}$	Representative value $k_{E,u}$, $k_{E,s}$	Return period
Ultimate	a) High	1.5〜2.0	0.4	500 years
	b) Normal	1.0		
	c) Low	0.4〜0.8		
Serviceability	a) High	1.5〜3.0	0.08	20 years
	b) Normal	1.0		
	c) Low	0.4〜0.8		

Table 3.2: Example 2 for load factors and representative values

Limit state	Degree of importance	Load factor $\gamma_{E,u}$, $\gamma_{E,s}$	Representative value $k_E = k_{E,u} = k_{E,s}$	Return period
Ultimate	a) High	3.0〜4.0	0.2	100 years
	b) Normal	2.0		
	c) Low	0.8〜1.6		
Serviceability	a) High	0.6〜1.2		
	b) Normal	0.4		
	c) Low	0.16〜0.32		

(2) Structural factor

The structural factor k_D is to reduce design seismic forces and shear forces, taking into account the ductility, acceptable deformation, restoring force characteristics and overstrength (or overcapacity) of the structure. The factor can be divided into two factors, namely $k_{D\mu}$ and k_{Ds} and expressed as the product of them, where $k_{D\mu}$ is related to ductility, acceptable deformation and restoring force characteristics, whereas k_{Ds} is related to overstrength.

Recent studies indicate that k_D also depends on the natural period of vibration of the structure and the possible reduction in strength remains minimal for structures having a shorter fundamental natural period. k_{Ds} is a function of the difference between the actual strength and calculated strength and varies according to the method of strength calculation. Quantification of these factors is a matter of debate, and one generic term k_D has been adopted in most codes.

The structural factor k_D may be, for example,

- 1/5 to 1/3 for systems with excellent ductility,
- 1/3 to 1/2 for systems with medium ductility,
- 1/2 to 1 for systems with poor ductility.

These values of k_D are under continuing investigation and may take other values in some circumstances.

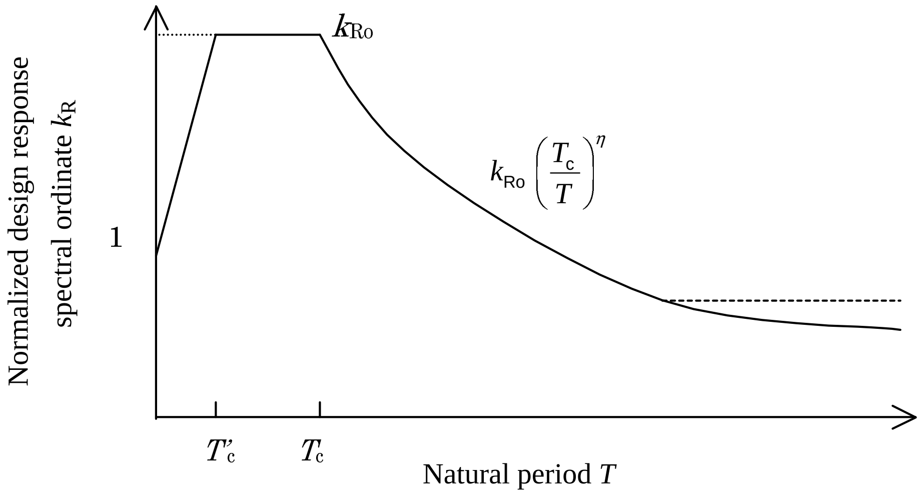


Figure 3.1: Normalized design response spectrum

(3) Normalized design response spectrum

The normalized design response spectrum can be interpreted as an acceleration spectrum normalized by the maximum ground acceleration for design purpose. It may be of the form as illustrated in Fig.3.1. In the figure, k_R is the ordinate of the normalized design response spectrum, and k_{Ro} is a factor dependent on the soil profile and the characteristics of the structure, e.g. the damping of the structure.

For a structure with a damping ratio of 0.05 resting on average quality soil, k_{Ro} may be taken as 2 to 3. T is the fundamental natural period of the structure, T_c and T'_c are the corner periods related to the soil condition, and η is an exponent that can vary from 1/3 to 1. T'_c may be taken as 1/5 to 1/2 of T_c .

For example, T_c can be taken as

- 0.3 to 0.5 (s) for stiff and hard soil conditions,
- 0.5 to 0.8 (s) for intermediate soil conditions,
- 0.8 to 1.2 (s) for loose and soft soil conditions.

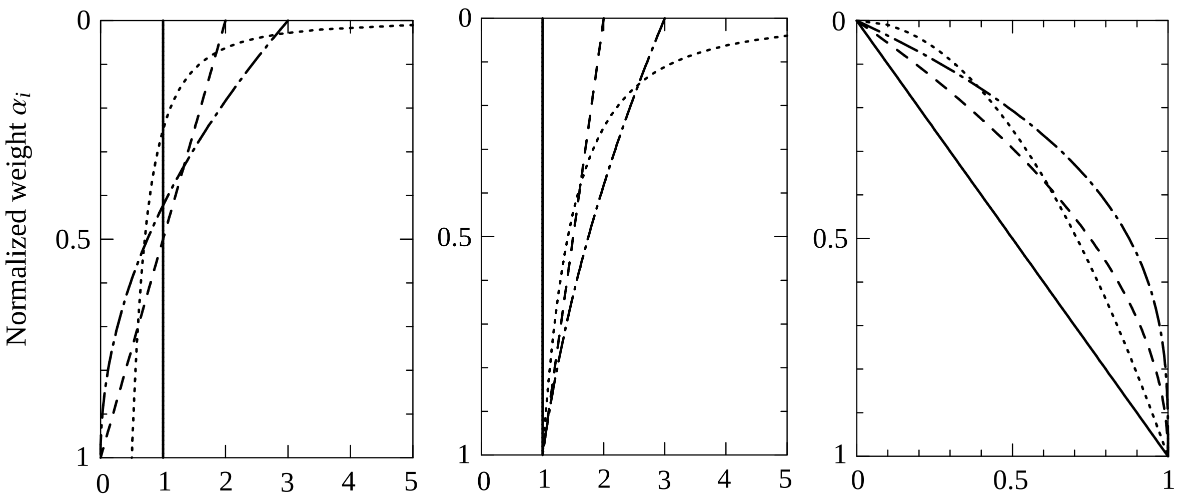
For structures with a fundamental natural period shorter than T'_c , it is recommended to use $k_R = k_{Ro}$ as indicated by the dotted line in Fig.3.1, because of the uncertainties in ground motion characteristics and the unconservative estimation of structural factor k_D in this range. For determining forces at longer periods, it is recommended that a lower limit be considered as indicated by the dashed line in Fig.3.1. The value of this level may be taken as 1/3 to 1/5 of k_{Ro}.

(4) Seismic force distribution factor and seismic shear distribution factor

General characteristics of distributions of seismic force parameters are as follows.

- For very low and stiff buildings, whole parts from the top to the base move along with the ground motion. In this case, the distribution of seismic forces is unform and seismic shear forces increases linearly from the top to the base (uniform distribution of seismic forces, see solid lines of Fig.3.2).

- For low-rise buildings, the distribution of seismic forces becomes similar to the inverted triangle and the distribution of seismic shear forces is assumed to be a parabola whose vertex locates at the base (inverted triangle distribution of seismic forces, see dashed curves of Fig.3.2).
- For high-rise buildings, seismic forces at the upper part become larger because of a higher mode effect. If the building is assumed to be a uniform shear type elastic body fixed at the base and to be subjected to white noise excitation, the distribution of seismic shear forces become a parabola whose vertex locates at the top (distribution for shear type structure subjected to white noise excitation, see dotted curves of Fig.3.2).



a) Seismic force distribution factor $k_{F,i}$ b) Seismic shear distribution factor $k_{V,i}$ c) Normalized seismic shear force $V_{E,i}$

Figure 3.2: Distribution of seismic force parameters

Taking into account the above characteristics of seismic force parameters, the seismic force distribution factor, $k_{F,i}$, can be determined by

$$k_{F,i} = \frac{F_{G,i} h_i^{\nu}}{\sum_{j=1}^{n} F_{G,j} h_j^{\nu}} \tag{3.5}$$

where, $F_{G,i}$ is the gravity load of the structure at the i-th level, h_i is the height above the base to the i-th level, n is the number of levels above the base. The exponent ν may be taken as follows:

- $\nu = 0$ for very low buildings (up to two-story buildings), or structures for which $T \leqq 0.2$ (s),
- $\nu = 0$ to 1 for low-rise buildings (three to five-story buildings), or structures for which 0.2 (s) $< T \leqq 0.5$ (s),
- $\nu = 1$ to 2 for intermediate buildings, or structures for which 0.5 (s) $< T \leqq 1.5$ (s),
- $\nu = 2$ for high-rise buildings (higher than 50 meters or more than fifteen-story buildings), or structures for which $T > 1.5$ (s).

Distributions of seismic force parameters given by Eq.(3.5) are shown as solid lines in Fig.3.2 for $\nu = 0$, as the dashed curves for $\nu = 1$, and as the dash-dotted curves for $\nu = 2$.

Since Eq.(3.5) does not give an appropriate distribution for high-rise buildings, the seismic force distribution factor, $k_{\mathrm{F},i}$, for high-rise buildings can be determined by

$$k_{\mathrm{F},n} = \rho \tag{3.6}$$

$$k_{\mathrm{F},i} = (1-\rho)\frac{F_{\mathrm{G},i}h_i}{\sum_{j=1}^{n} F_{\mathrm{G},j}h_j} \tag{3.7}$$

where, ρ is the factor to give a concentrated force at the top and is approximately equal to 0.1.

Since Eqs.(3.6) and (3.7) do not always give an appropriate distribution and a concentrated force at the top is not practical for buildings with setbacks, it is preferable using the seismic shear distribution factor, $k_{\mathrm{V},i}$, instead of seismic force distribution factor $k_{\mathrm{F},i}$. The factor $k_{\mathrm{V},i}$ is interpreted as the shear factor of the i-th level normalized by the base shear factor.

The seismic shear distribution factor, $k_{\mathrm{V},i}$, can be determined by

$$k_{\mathrm{V},i} = 1 + k_1(1-\alpha_i) + k_2\left(\frac{1}{\sqrt{\alpha_i}} - 1\right) \tag{3.8}$$

where, k_1 and k_2 are factors from 0 to 1 and determined mainly by the height or the fundamental natural period of the structure, and α_i is the normalized weight which is given by

$$\alpha_i = \frac{\sum_{j=i}^{n} F_{\mathrm{G},j}}{\sum_{j=1}^{n} F_{\mathrm{G},j}} \tag{3.9}$$

The normalized weight is used instead of the height of levels above the base, because the normalized weight is more convenient and rational to express the distribution of seismic force parameters. The ordinate in Fig.3.2a-c) is the normalized weight.

Distributions of seismic force parameters given by Eq.(3.8) are shown as solid lines in Fig.3.2 for $k_1 = 0$ and $k_2 = 0$ (uniform distribution of seismic forces), as the dashed curves for $k_1 = 1$ and $k_2 = 0$ (inverted triangular distribution of seismic forces), and as the dotted curves for $k_1 = 0$ and $k_2 = 1$ (distribution for shear type structure subjected to white noise excitation). Therefore, the factors k_1 and k_2 may be taken as follows:

- $k_1 \approx 0$ and $k_2 \approx 0$ for very low buildings,
- $k_1 \approx 1$ and $k_2 \approx 0$ for low-rise buildings,
- $k_1 \approx 0.5$ and $k_2 \approx 0.5$ for intermediate buildings,
- $k_1 \approx 0$ and $k_2 \approx 1$ for high-rise buildings.

(5) Components of seismic action

Two horizontal components of the earthquake ground motion influence the total seismic actions on the structure, e.g. torsional moment of the structure with two-directional eccentricity, and axial force of corner columns.

When the two horizontal components of seismic action are denoted as E_{x} and E_{y}, sometimes the SRSS method is applied to obtain the total design seismic action. The method, however, often underestimates the maximum response. Therefore it is recommended to use the following quadratic combination:

$$E = \sqrt{E_{\mathrm{x}}^2 + 2\epsilon E_{\mathrm{x}} E_{\mathrm{y}} + E_{\mathrm{y}}^2} \tag{3.10}$$

While the factor ϵ can be from -1 to 1, ϵ may empirically be taken as 0 to 0.3.

(6) Torsional moments

The torsional moment of the i-th level of the structure, M_i, which is usually calculated in each direction of orthogonal axes, x and y, may be determined by

$$M_i = V_i\, e_i \tag{3.11}$$

where V_i is the seismic shear force of the i-th level:

$$V_i = \sum_{j=i}^{n} F_j \tag{3.12}$$

where n is the number of levels above the base, and e_i is one of the following two values: the eccentricity between the centers of mass and stiffness multiplied by a dynamic magnification factor plus the incidental eccentricity, and the eccentricity between the centers of mass and stiffness minus the incidental eccentricity.

(7) Dynamic response

a) Response spectrum analysis

When natural frequencies of different modes are not closely spaced to each other, combination to estimate the maximum response quantity may generally be performed usin the following formula (SRSS method):

$$S = \sqrt{\sum_{i=1}^{n} S_i^2} \tag{3.13}$$

where, S is the maximum response quantity under consideration, and S_i is the maximum response quantity in the i-th mode of vibration.

When natural frequencies of two or more modes are closely sapced, the combination may be performed using the following formula (CQC method):

$$S = \sqrt{\sum_{i=1}^{n} \sum_{k=1}^{n} S_i\, \rho_{i,k}\, S_k} \tag{3.14}$$

$$\rho_{i,k} = \frac{8\sqrt{\xi_i\, \xi_k}(\zeta_i + \chi\, \zeta_k)\chi^{3/2}}{(1-\chi^2)^2 + 4\,\xi_i\, \xi_k \chi\,(1+\chi^2) + 4(\xi_i^2 + \xi_k^2)\chi^2} \tag{3.15}$$

where, ζ_i and ζ_k are the damping ratios for the i-th and k-th mode, and χ is the ratio of the i-th mode natural frequency to the k-th mode natural frequency.

b) Time history analysis

(Types of models of the structure) Models of the structure should be chosen based on the purpose of the analysis. Basically, the models used in the time history analysis are the same as those used in the response spectrum analysis.

In many cases one-dimensional lumped mass shear models are used for low- to medium-rise buildings, where a lumped mass represents the mass of each story and lateral stiffness of storys are independent. For high-rise buildings and slender structures, shear-flexural models are recommended to be used, taking into account the axial deformation of columns or the flexual deformation of overall bending of the structure.

In general, models fixed at the base may be employed. When the effects of ground compliance are to be considered, sway-rocking models may be employed. Soil-structure interaction models may be used when earthquake ground motions are defined at the bedrock.

(Restoring force characteristics) Although any rational restoring force characteristics are accepted, in principle, they should be elasto-plastic. Elastic models may be accepted where response of the elasto-plastic range is not expected or quite limited. In general, bilinear or trilinear restoring force characteristic models are used for steel elements. For reinforced concrete elements, degrading trilinear models are used, since the stiffness degradation of those elements can not be neglected.

(Input earthquake motions) When recorded earthquake ground motions are used as input ground motions, they shoud be scaled appropriately. In general, the acceleration records are scaled to have the same maximum velocity in order to avoid the fluctuation in the response.

Simulated earthquake ground motions may be established either at the ground surface or at the bedrock. It is more rational to establish the simulated earthquake ground motions at the bedrock which can be used directly in the soil-structure interaction model analysis. When simulated earthquake ground motions are set up at the ground surface, they should reflect the dynamic characteristics of the soil.

(8) Damping ratio

Damping in the structures consists of internal damping of structural elements, hysteretic damping related to elasto-plastic restoring force characteristics, damping due to non-structural elements, and damping due to energy dissipation into the ground.

In general, these types of damping (except the hysteretic damping) are represented by viscous damping coefficients in dynamic analysis.

For design purposes, the damping ratio for the fundamental mode of regular structures is offen taken as 0.02 to 0.05 depending on the types of construction. One of the classical damping matrices is the Rayleigh damping, for which the damping matrix $[C]$ is given as follows:

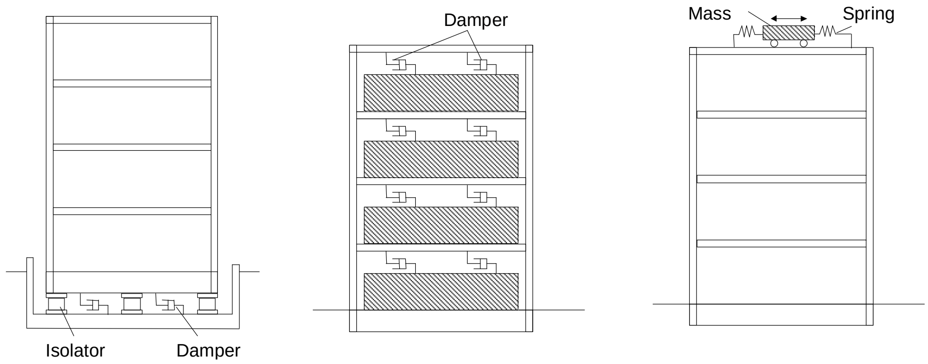
$$[C] = a_0[M] + a_1[K] \tag{3.16}$$

where, $[M]$ is the mass matrix, $[K]$ is the stiffness matrix, and a_0 and a_1 are the coefficients to be determined depending upon the damping of two different modes.

The above damping matrix may not provide appropriate damping ratios for modes other than the two modes considered for determining the coefficients a_0 and a_1. In such cases, other damping matrices in which modal damping ratios can be specified individually for multiple modes may be applied.

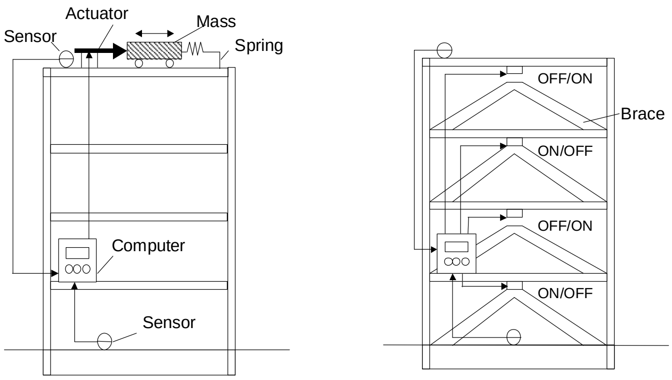
(9) Response control systems

Recently response control systems, including seismic isolation, have been gradually applied to various structures. Some examples are illustrated in Fig.3.3.



1-a) Base isolation 1-b) Energy absorption 1-c) Tuned mass damper

1) Passive systems



2-a) Mass effect mechanism 2-b) Stiffness control

2) Active systems

Figure 3.3: Response control systems

(10) Paraseismic influences

Sources of paraseismic influences are classified as follows: underground explosions, underground shocks from exploited (also abandoned) mines, over ground impacts and schocks (e.g. pile driving), trafic vibration transmitted through ground to buildings (from surface motorways, streets, railway lines, underground railways), and other sources such as industry activities and machines.

3.2 European Seismic Code: Eurocode 8

1) Eurocode

Eurocode is the European standard for the design of construction works. It consists of the following standards:

EN1990: Eurocode 0 Basis of structural design
EN1991: Eurocode 1 Actions on structures
EN1992: Eurocode 2 Design of concrete structures
EN1993: Eurocode 3 Design of steel structures
EN1994: Eurocode 4 Design of composite steel and concrete structures
EN1995: Eurocode 5 Design of timber structures
EN1996: Eurocode 6 Design of masonry structures
EN1997: Eurocode 7 Geotechnical design
EN1998: Eurocode 8 **Design of structures for earthquake resistance**[6]

2) EN 1998: Eurocode 8

EN 1998: Eurocode 8 consists of the following parts:

EN1998-1: **Design of buildings and civil engineering works in seismic region**
EN1998-2: Specific provisions relevant to bridges
EN1998-3: Provisions for the seismic assessment and retrofitting of existing buildings
EN1998-4: Specific provisions relevant to silos, tanks and pipelines
EN1998-5: Specific provisions relevant to foundations, retaining structures and geotechnical aspects
EN1998-6: Specific provisions relevant to bridges towers, masts and chimneys

3) EN 1998-1: Part 1 of Eurocode 8

EN 1998-1: Part 1 of Eurocode 8 consists of the following sections:

Section 1 **General**
Section 2 **Performance requirements and compliance criteria**
Section 3 **Ground conditions and seismic action**
Section 4 **Design of buildings**
Section 5 Specific rules for concrete buildings
Section 6 Specific rules for steel buildings
Section 7 Specific rules for composite steel-concrete buildings
Section 8 Specific rules for timber buildings
Section 9 Specific rules for masonry buildings
Section 10 Base isolation

EN1998-1 from Section 1 to Section 4 is summarized as follows.

Section 1 General

EN 1998 (Eurocode 8) applies to the design and construction of buildings and civil engineering works in seismic regions. Its purpose is to ensure that in the event of earthquakes:

- human lives are protected;
- damage is limited; and
- structures important for civil protection remain operational.

Section 2 Performance Requirements and Compliance Criteria

a) Fundamental requirements

Structures in seismic regions shall be designed and constructed in such a way that the following requirements are met, each with an adequate degree of reliability.

– Non-collapse requirement.

The design seismic action is expressed in terms of: a) the reference seismic action associated with a reference probability of exceedance, P_{NCR}, in 50 years or a reference return period, T_{NCR}, and b) the importance factor γ_{I}.

Note) The recommended values are $P_{\mathrm{NCR}} = 10\,\%$ and $T_{\mathrm{NCR}} = 475$ years.

– Damage limitation requirement.

The structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself. The seismic action to be taken into account for the “damage limitation requirement” has a probability of exceedance, P_{DCR}, in 10 years or a return period, T_{DCR}.

Note) The recommended values are $P_{\mathrm{DCR}} = 10\,\%$ and $T_{\mathrm{DCR}} = 95$ years.

b) Compliance criteria

In order to satisfy the fundamental requirements the following limit states shall be checked:

– ultimate limit states;
– damage limitation states.

(1) Ultimate limit state

It shall be verified that the structural system has the resistance and energy dissipation capacity specified in the relevant Parts of EN 1998. It shall be verified that under the design seismic action the behaviour of nonstructural elements does not present risks to persons and does not have a detrimental effect on the response of the structural elements.

(2) Damage limitation state

An adequate degree of reliability against unacceptable damage shall be ensured by satisfying the deformation limits or other relevant limits defined in the relevant Parts of EN 1998.

Section 3 Ground Conditions and Seismic Action

a) Ground conditions

Appropriate investigations shall be carried out in order to identify the ground conditions.

Ground types A, B, C, D, and E, described by the stratigraphic profiles and parameters given in Table 3.3, may be used to account for the influence of local ground conditions on the seismic action.

Table 3.3: Ground types

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface	> 800	–	–
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth	360～800	> 50	> 250
C	Deep deposit of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters	180～360	15～50	70～250
D	Deposit of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800(m/s).			
S_1	Deposits consisting, or containing a layer at least 10 m thick, or soft clays/silts with a high plasticity index (PI > 40) and high water content.	< 100 (indicative)	–	10～20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A ～E or S_1.			

Note) v_s: shear wave velocity, $v_{s,30}$: average shear wave velocity in upper 30m soil, N_{SPT}: Standard Penetration Test blow-count, c_u: undrained shear strength of soil

b) Seismic action

(1) Seismic zones

National territories shall be subdivided by the National Authorities into seismic zones, depending on the local hazard. The hazard is described in terms of a single parameter, i.e. the value of the reference peak ground acceleration on type A ground, a_{gR}.

(2) Basic representation of the seismic action

(i) General

The earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum, henceforth called an "elastic response spectrum".

(ii) Horizontal elastic response spectrum

For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by (see Fig.3.4):

$$S_e(T) = \begin{cases} a_g \cdot S \cdot \left[1 + \dfrac{T}{T_B}(\eta \cdot 2.5 - 1)\right] & (0 \leqq T \leqq T_B) \\ a_g \cdot S \cdot \eta \cdot 2.5 & (T_B \leqq T \leqq T_C) \\ a_g \cdot S \cdot \eta \cdot 2.5\left[\dfrac{T_C}{T}\right] & (T_C \leqq T \leqq T_D) \\ a_g \cdot S \cdot \eta \cdot 2.5\left[\dfrac{T_C T_D}{T^2}\right] & (T_D \leqq T \leqq 4\text{s}) \end{cases} \quad (3.17)$$

where, T is the vibration period of a linear single-degree-of-freedom system; a_g is the design ground acceleration on type A ground ($a_g = \gamma_I a_{gR}$); T_B is the lower limit of the period of the constant spectral acceleration branch; T_C is the upper limit of the period of the constant spectral acceleration branch; T_D is the value defining the beginning of the constant displacement response range of the spectrum; S is the soil factor; η is the damping correction factor with a reference value of $\eta = 1$ for 5 % viscous damping.

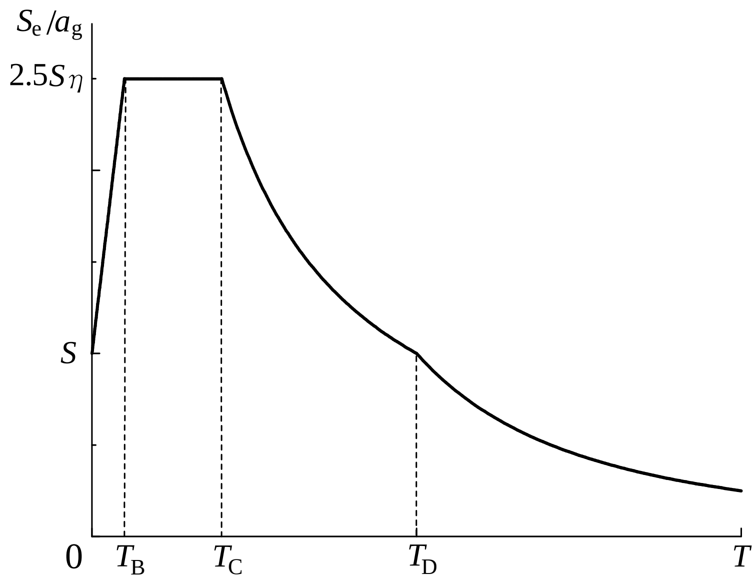


Figure 3.4: Shape of the elastic response spectrum

The values of the periods T_B, T_C and T_D and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type (see Table 3.4, Table 3.5, Fig.3.5 and Fig.3.6).

Note) If deep geology is not accounted for, the recommended choice is the use of two types of spectra: Type 1 and Type 2. If the earthquakes that contribute most to the seismic hazard defined for the site for

Table 3.4: Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.20	0.6	2.0
D	1.35	0.20	0.8	2.0
E	1.4	0.15	0.5	2.0

Table 3.5: Values of the parameters describing the recommended Type 2 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1.0	0.05	0.25	1.2
B	1.35	0.05	0.25	1.2
C	1.5	0.10	0.25	1.2
D	1.8	0.10	0.30	1.2
E	1.6	0.05	0.25	1.2

the purpose of probabilistic hazard assessment have a surface-wave magnitude, M_S, not greater than 5,5, it is recommended that the Type 2 spectrum is adopted.

The value of the damping correction factor η may be determined by:

$$\eta = \sqrt{10/(5+\xi)} \geqq 0.55 \tag{3.18}$$

where, ξ is the viscous damping ratio of the structure, expressed as a percentage.

The elastic displacement response spectrum, $S_{De}(T)$, shall be obtained by direct transformation of the elastic acceleration response spectrum, $S_e(T)$, using the following expression:

$$S_{De}(T) = S_e(T)\left[\frac{T}{2\pi}\right]^2 \tag{3.19}$$

(iii) Vertical elastic response spectrum

The vertical component of the seismic action shall be represented by an elastic response spectrum, $S_{ve}(T)$, derived using following expressions (see Table 3.6).

$$S_{ve}(T) = \begin{cases} a_{vg} \cdot \left[1 + \dfrac{T}{T_B}(\eta \cdot 3.0 - 1)\right] & (0 \leqq T \leqq T_B) \\ a_{vg} \cdot \eta \cdot 3.0 & (T_B \leqq T \leqq T_C) \\ a_{vg} \cdot \eta \cdot 3.0\left[\dfrac{T_C}{T}\right] & (T_C \leqq T \leqq T_D) \\ a_{vg} \cdot \eta \cdot 3.0\left[\dfrac{T_C T_D}{T^2}\right] & (T_D \leqq T \leqq 4\text{s}) \end{cases} \tag{3.20}$$

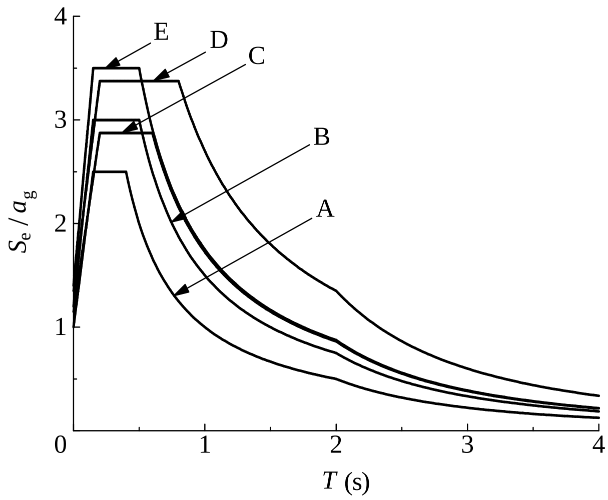


Figure 3.5: Recommended Type 1 elastic response spectra for ground types A to E (5 % damping)

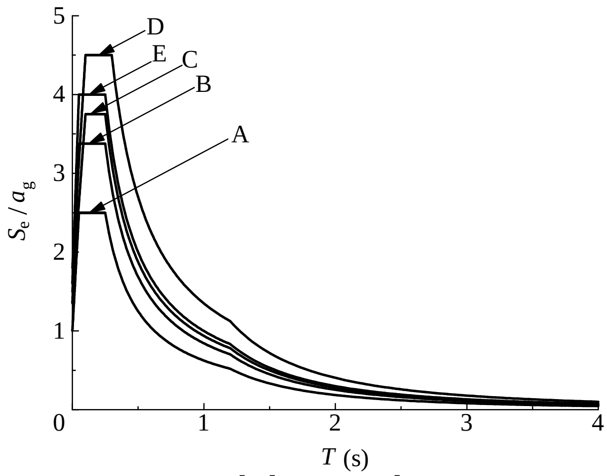


Figure 3.6: Recommended Type 2 elastic response spectra for ground types A to E (5 % damping)

(iv) Design ground displacement

The design ground displacement d_g, corresponding to the design ground acceleration, may be estimated by:

$$d_g = 0.025\, a_g\, S\; T_C\, T_D \tag{3.21}$$

(v) Design spectrum for elastic analysis

The capacity of structural systems to resist seismic actions in the non-linear range generally permits their design for resistance to seismic forces smaller than those corresponding to a linear elastic response.

To avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behaviour of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a response spectrum reduced with respect to the elastic one, henceforth called a "design spectrum". This reduction is accomplished

Table 3.6: Recommended values of parameters describing the vertical elastic response spectra

Spectrum	a_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0.90	0.05	0.15	1.0
Type 2	0.45	0.05	0.15	1.0

by introducing the behaviour factor q.

The behaviour factor q is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5 % viscous damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure.

For the horizontal components of the seismic action the design spectrum, $S_d(T)$, shall be defined by:

$$S_d(T) = \begin{cases} a_g \cdot S\left[\frac{2}{3} + \frac{T}{T_B}(\frac{2.5}{q} - \frac{2}{3})\right] & (0 \leqq T \leqq T_B) \\ a_g \cdot S \cdot \frac{2.5}{q} & (T_B \leqq T \leqq T_C) \\ a_g \cdot S \cdot \eta \cdot \frac{2.5}{q}\left[\frac{T_C}{T}\right] \geqq \beta \cdot a_g & (T_C \leqq T \leqq T_D) \\ a_g \cdot S \cdot \eta \cdot \frac{2.5}{q}\left[\frac{T_C T_D}{T^2}\right] \geqq \beta \cdot a_g & (T_D \leqq T) \end{cases} \tag{3.22}$$

where, β is the lower bound factor for the horizontal design spectrum.

Note) The recommended value for β is 0.2.

(3) Alternative representations of the seismic action

The seismic motion may also be represented in terms of ground acceleration time-histories and related quantities (velocity and displacement).

Depending on the nature of the application and on the information actually available, the description of the seismic motion may be made by using artificial accelerograms and recorded or simulated accelerograms.

(Artificial accelerograms) Artificial accelerograms shall be generated so as to match the elastic response spectra for 5 % viscous damping ($\xi = 5$ %).

When site-specific data are not available, the minimum duration T_s of the stationary part of the accelerograms should be equal to 10 (s).

(Recorded or simulated accelerograms) Recorded accelerograms, or accelerograms generated through a physical simulation of source and travel path mechanisms, may be used, provided that the samples used are adequately qualified with regard to the seismogenetic features of the sources and to the soil conditions appropriate to the site, and their values are scaled to the value of $a_g S$ for the zone under consideration.

Section 4 Design of Buildings

a) Characteristics of earthquake resistant buildings

(1) Basic principles of conceptual design

In seismic regions the aspect of seismic hazard shall be taken into account in the early stages of the conceptual design of a building, thus enabling the achievement of a structural system which, within acceptable costs, satisfies the fundamental requirements.

The guiding principles governing this conceptual design are:

- structural simplicity;
- uniformity, symmetry and redundancy;
- bi-directional resistance and stiffness;
- torsional resistance and stiffness;
- diaphragmatic behaviour at storey level;
- adequate foundation

(2) Primary and secondary seismic members

A certain number of structural members (e.g. beams and/or columns) may be designated as "secondary" seismic members (or elements), not forming part of the seismic action resisting system of the building. The strength and stiffness of these elements against seismic actions shall be neglected. They do not need to conform to the requirements of Section 5 to Section 9[†]. Nonetheless these members and their connections shall be designed and detailed to maintain support of gravity loading when subjected to the displacements caused by the most unfavourable seismic design condition. Due allowance of second-order effects (P-Δ effects) should be made in the design of these members.

All structural members not designated as being secondary seismic members are taken as being primary seismic members. They are taken as being part of the lateral force resisting system, should be modelled in the structural analysis and designed and detailed for earthquake resistance in accordance with the rules of Section 5 to Section 9.

(3) Criteria for structural regularity

For the purpose of seismic design, building structures are categorised into being regular or non-regular. This distinction has implications for the following aspects of the seismic design (see Table 3.7):

- the structural model, which can be either a simplified planar model or a spatial model;
- the method of analysis, which can be either a simplified response spectrum analysis (lateral force procedure) or a modal one;
- the value of the behaviour factor q, which shall be decreased for buildings non-regular in elevation.

(4) Importance classes and importance factors

Buildings are classified in 4 importance classes, depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse (see Table 3.8).

[†]These sections are not included in this summary.

Table 3.7: Consequences of structural regularity on seismic analysis and design

Regularity		Allowed simplification		Behavior factor q
Plan	Elevation	Model	Linear-elastic analysis	(for linear analysis)
Yes	Yes	Planar	Lateral force*	Reference value
Yes	No	Planar	Modal**	Decreased value
No	Yes	Spatial	Lateral force*	Decreased value
No	No	Spatial	Modal**	Decreased value

*Lateral force method of analysis, **Modal response spectrum analysis

Table 3.8: Importance classes for buildings

Important class	Buildings	Recommended values of γ_{I}
I	Building of minor importance for public safety, e.g. agricultural buildings, etc.	0.8
II	Ordinary buildings, not belonging in other categories.	1.0
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions, etc.	1.2
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.	1.4

b) Structural analysis

(1) Modelling

The model of the building shall adequately represent the distribution of stiffness and mass in it so that all significant deformation shapes and inertia forces are properly accounted for under the seismic action considered. In the case of non-linear analysis, the model shall also adequately represent the distribution of strength.

The model should also account for the contribution of joint regions to the deformability of the building, e.g. the end zones in beams or columns of frame type structures. Non-structural elements, which may influence the response of the primary seismic structure, should also be accounted for.

The deformability of the foundation shall be taken into account in the model, whenever it may have an adverse overall influence on the structural response.

Note) Foundation deformability (including the soil-structure interaction) may always be taken into account, including the cases in which it has beneficial effects.

(2) Accidental torsional effects

In order to account for uncertainties in the location of masses and in the spatial variation of the seismic motion, the calculated centre of mass at each floor i shall be considered as being displaced from its nominal location in each direction by an accidental eccentricity:

$$e_{ai} = \pm 0.05 L_i \tag{3.23}$$

where, e_{ai} is the accidental eccentricity of storey mass i from its nominal location, applied in the same direction at all floors; L_i is the floor-dimension perpendicular to the direction of the seismic action.

(3) Methods of analysis

(i) General

The seismic effects and the effects of the other actions included in the seismic design situation may be determined on the basis of the linear-elastic behaviour of the structure.

Depending on the structural characteristics of the building, one of the following two types of linear-elastic analysis may be used:

a) "lateral force method of analysis" for buildings meeting the conditions;
b) "modal response spectrum analysis", which is applicable to all types of buildings.

As an alternative to a linear method, a non-linear method may also be used, such as:

c) non-linear static (pushover) analysis;
d) non-linear time history (dynamic) analysis,

provided that the conditions specified are satisfied.

Linear-elastic analysis may be performed using two planar models, one for each main horizontal direction, if the criteria for regularity in plan are satisfied .

Whenever a spatial model is used, the design seismic action shall be applied along all relevant horizontal directions (with regard to the structural layout of the building) and their orthogonal horizontal directions. For buildings with resisting elements in two perpendicular directions these two directions shall be considered as the relevant directions.

(ii) Lateral force method of analysis

This type of analysis may be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction.

(Base shear force) The seismic base shear force F_b, for each horizontal direction in which the building is analysed, shall be determined:

$$F_b = S_d(T_1)\, m\, \lambda \tag{3.24}$$

where, $S_d(T_1)$ is the ordinate of the design spectrum at period T_1; T_1 is the fundamental period of vibration of the building for lateral motion in the direction considered; m is the total mass of the building, above the foundation or above the top of a rigid basement; λ is the correction factor, the value of which is equal to: $\lambda = 0.85$ if $T_1 \leqq 2T_c$ and the building has more than two storeys, or $\lambda = 1.0$ otherwise.

For the determination of the fundamental period of vibration T_1 of the building, expressions based on methods of structural dynamics (for example the Rayleigh method) may be used.

For buildings with heights of up to 40 m the value of T_1(s) may be approximated by:

$$T_1 = C_t H^{3/4} \tag{3.25}$$

where, C_t is 0.085 for moment resistant space steel frames, 0.075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0.050 for all other structures; H(m) is the height of the building from the foundation or from the top of a rigid basement.

Alternatively, for structures with concrete or masonry shear walls the value C_t in Eq.(3.25) may be taken as being

$$C_t = 0.075/\sqrt{A_c} \tag{3.26}$$

where,

$$A_c = \Sigma[A_i(0.2 + (l_{wi}/H))^2] \tag{3.27}$$

A_c (m^2) is the total effective area of the shear walls in the first storey of the building; A_i (m^2) is the effective cross-sectional area of the shear wall i in the first storey of the building; l_{wi} (m) is the length of the shear wall i in the first storey in the direction parallel to the applied forces with the restriction that l_{wi}/H should not exceed 0.9.

Alternatively, the estimation of T_1(s) may be made by:

$$T_1 = 2\sqrt{d} \tag{3.28}$$

where, d (m) is the lateral elastic displacement of the top of the building due to the gravity loads applied in the horizontal direction.

(Distribution of the horizontal seismic forces) The fundamental mode shapes in the horizontal directions of analysis of the building may be calculated using methods of structural dynamics or may be approximated by horizontal displacements increasing linearly along the height of the building.

The seismic action effects shall be determined by applying, to the two planar models, horizontal forces F_i to all storeys.

$$F_i = F_b \frac{s_i m_i}{\Sigma s_j m_j} \tag{3.29}$$

where, F_i is the horizontal force acting on storey i; F_b is the seismic base shear in accordance with Eq.(3.24); s_i, s_j are the displacements of masses m_i, m_j in the fundamental mode shape; m_i, m_j are the storey masses.

When the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height, the horizontal forces F_i should be taken as follows:

$$F_i = F_b \frac{z_i m_i}{\Sigma z_j m_j} \tag{3.30}$$

where, z_i, z_j are the heights of the masses m_i, m_j above the level of application of the seismic action (foundation or top of a rigid basement).

(Torsional effects) If the lateral stiffness and mass are symmetrically distributed in plan and unless the accidental eccentricity is taken into account by a more exact method, the accidental torsional effects may be accounted for by multiplying the action effects in the individual load resisting elements by a factor δ given by

$$\delta = 1 + 0.6\frac{x}{L_e} \tag{3.31}$$

where, x is the distance of the element under consideration from the centre of mass of the building in plan, measured perpendicularly to the direction of the seismic action considered; L_e is the distance between the two outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action considered.

If the analysis is performed using two planar models, one for each main horizontal direction, torsional effects may be determined by doubling the accidental eccentricity $e_{\mathrm{a}i}$ of Eq.(3.23) and applying Eq.(3.31) with factor 0.6 increased to 1.2.

(iii) Modal response spectrum analysis

This type of analysis shall be applied to buildings which do not satisfy the conditions for applying the lateral force method of analysis.

The response of all modes of vibration contributing significantly to the global response shall be taken into account.

The requirements specified above may be deemed to be satisfied if either of the following can be demonstrated:

– the sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure;
– all modes with effective modal masses greater than 5 % of the total mass are taken into account.

When using a spatial model, the above conditions should be verified for each relevant direction.

(Combination of modal responses) The response in two vibration modes i and j (including both translational and torsional modes) may be taken as independent of each other, if their periods T_i and T_j satisfy (with $T_j \leqq T_i$) the following condition:

$$T_j \leqq 0.9\, T_i \tag{3.32}$$

Whenever all relevant modal responses may be regarded as independent of each other, the maximum value E_E of a seismic action effect may be taken as:

$$E_\mathrm{E} = \sqrt{\Sigma E_{\mathrm{E}i}^2} \tag{3.33}$$

where, $E_{\mathrm{E}i}$ is the value of this seismic action effect due to the vibration mode i.

If Eq.(3.32) is not satisfied, more accurate procedures for the combination of the modal maxima, such as the “Complete Quadratic Combination” shall be adopted.

(iv) Non-linear methods

The mathematical model used for elastic analysis shall be extended to include the strength of structural elements and their post-elastic behaviour.

The seismic action shall be applied in both positive and negative directions and the maximum seismic effects as a result of this shall be used.

(Non-linear static (pushover) analysis) Pushover analysis is a non-linear static analysis carried out under conditions of constant gravity loads and monotonically increasing horizontal loads.

At least two vertical distributions of the lateral loads should be applied:

– a “uniform” pattern, based on lateral forces that are proportional to mass regardless of elevation (uniform response acceleration);
– a “modal” pattern, proportional to lateral forces consistent with the lateral force distribution in the direction under consideration determined in elastic analysis.

(Non-linear time-history analysis) The time-dependent response of the structure may be obtained through direct numerical integration of its differential equations of motion, using the accelerograms to represent the ground motions.

The structural element models should be supplemented with rules describing the element behaviour under post-elastic unloading-reloading cycles. These rules should realistically reflect the energy dissipation in the element over the range of displacement amplitudes expected in the seismic design situation.

(v) Combination of the effects of the components of the seismic action

(Horizontal components of the seismic action) In general the horizontal components of the seismic action shall be taken as acting simultaneously.

The combination of the horizontal components of the seismic action may be accounted for as follows.

a) The structural response to each component shall be evaluated separately, using the combination rules for modal responses.
b) The maximum value of each action effect on the structure due to the two horizontal components of the seismic action may then be estimated by the square root of the sum of the squared values of the action effect due to each horizontal component.
c) More accurate models may be used for the estimation of the probable simultaneous values of more than one action effect due to the two horizontal components of the seismic action.

As an alternative to b) and c), the action effects due to the combination of the horizontal components of the seismic action may be computed using both of the two following combinations:

$$E_{\mathrm{Edx}}\text{“}+\text{”}0.30\,E_{\mathrm{Edy}} \tag{3.34}$$

$$0.30\,E_{\mathrm{Edx}}\text{“}+\text{”}E_{\mathrm{Edy}} \tag{3.35}$$

where, “+” implies “to be combined with”; E_{Edx} represents the action effects due to the application of the seismic action along the chosen horizontal axis x of the structure; E_{Edy} represents the action effects due to the application of the same seismic action along the orthogonal horizontal axis y of the structure.

The sign of each component in the above combinations shall be taken as being the most unfavourable for the particular action effect under consideration.

(Vertical component of the seismic action) If a_{vg} is greater than 0.25g (2.5m/s^2), the vertical component of the seismic action should be taken into account in the cases listed below:

– for horizontal or nearly horizontal structural members spanning 20 m or more;
– for horizontal or nearly horizontal cantilever components longer than 5 m;
– for horizontal or nearly horizontal pre-stressed components;
– for beams supporting columns;
– in base-isolated structures.

c) Safety verification

(1) Ultimate limit state

(i) Resistance condition

The following relation shall be satisfied for all structural elements including connections and the relevant non-structural elements:

$$E_{\mathrm{d}} \leqq R_{\mathrm{d}} \tag{3.36}$$

where, E_{d} is the design value of the action effect, due to the seismic design situation, including, if necessary, second order effects; and R_{d} is the corresponding design resistance of the element.

Second-order effects (P-Δ effects) need not be taken into account if the following condition is fulfilled in all storeys:

$$\theta = \frac{P_{\mathrm{tot}}\, d_{\mathrm{r}}}{V_{\mathrm{tot}}\, h} \leqq 0.10 \tag{3.37}$$

where, θ is the interstorey drift sensitivity coefficient; P_{tot} is the total gravity load at and above the storey considered in the seismic design situation; d_{r} is the design interstorey drift, evaluated as the difference of the average lateral displacements d_{s} at the top and bottom of the storey under consideration; V_{tot} is the total seismic storey shear; and h is the interstorey height.

If $0.1 < \theta \leqq 0.2$, the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1-\theta)$.

The value of the coefficient θ shall not exceed 0.3.

(ii) Global and local ductility condition

It shall be verified that both the structural elements and the structure as a whole possess adequate ductility, taking into account the expected exploitation of ductility, which depends on the selected system and the behaviour factor.

In multi-storey buildings formation of a soft storey plastic mechanism shall be prevented, as such a mechanism might entail excessive local ductility demands in the columns of the soft storey.

In frame buildings, including frame-equivalent ones, with two or more storeys, the following condition should be satisfied at all joints of primary or secondary seismic beams with primary seismic columns:

$$\Sigma M_{\mathrm{Rc}} \geqq 1.3\, \Sigma M_{\mathrm{Rb}} \tag{3.38}$$

where, ΣM_{Rc} is the sum of the design values of the moments of resistance of the columns framing the joint; and ΣM_{Rb} is the sum of the design values of the moments of resistance of the beams framing the joint.

(2) Damage limitation

The “damage limitation requirement” is considered to have been satisfied, if, under a seismic action having a larger probability of occurrence than the design seismic action corresponding to the “no-collapse requirement”, the interstorey drifts are limited in accordance with:

a) for buildings having non-structural elements of brittle materials attached to the structure:

$$d_{\mathrm{r}}\, \nu \leqq 0.005\, h \tag{3.39}$$

b) for buildings having ductile non-structural elements:

$$d_{\mathrm{r}}\,\nu \leqq 0.0075\,h \tag{3.40}$$

c) for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

$$d_{\mathrm{r}}\,\nu \leqq 0.010\,h \tag{3.41}$$

where, d_{r} is the design interstorey drift; h is the storey height; and ν is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement.

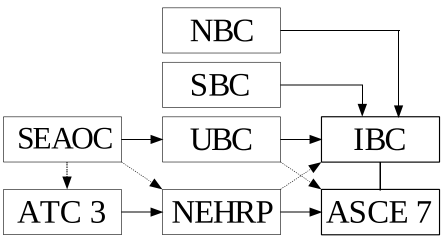
3.3 International Building Code (IBC) of U.S.A.

Introduction

International Building Code (IBC)[7] is the code to be used in U.S.A and also internationally. IBC was prepared combining mainly previous three model codes that had been used in U.S.A., i.e. Uniform Building Code (UBC) which had been used in western part, National Building Code (NBC) in eastern and northern parts and Standard Building Code (SBC) in southern part. The first edition of IBC was published in 1997, and it has been revised ever three years. First two editions included detailed requirements, but the 2003 and 2006 editions were revised to include only principal requirements, and refer to ASCE 7[8] "Seismic Design Requirements for Building Structures" for detailed requirements for seismic design.

IBC covers not only structural requirements but also interior finishings, environments, etc. concerning buildings. This summarizes seismic requirements of Chapter 16 "Structural design" of IBC 2006 edition.

As to the seismic codes of U.S.A., the first code was prepared by the committee in the Structural Engineers Association of California (SEAOC). The SEAOC code then transfered to UBC and then to IBC. The provision by Applied Technology Coiuncil (ATC) which is called as ATC 3[9] and the provisions by the National Earthquake Hazard Reduction Program (NEHRP) also influenced the IBC. The flowchart of Fig.3.7 schematically shows the relationships of these seismic codes.



SEAOC: Structural Engineers Association of California
ATC: Applied Technology Council
NBC: National Building Code
SBC: Standard Building Code
UBC: Unform Building Code
NEHRP: National Earthquake Hazard Reduction Program
IBC: International Building Code
ASCE: American Society of Civil Engineers

Figure 3.7: Flowchart of seismic codes in U.S.A.

1) Earthquake design data

The following information related to seismic loads shall be shown.

1. Seismic importance factor, I, and occupancy category
2. Mapped spectral response acceleration, S_{s} and S_{1}
3. Site class
4. Spectral response coefficient, S_{Ds} and S_{D1}
5. Seismic design category
6. Basic seismic-force-resisting system
7. Design base shear
8. Seismic response coefficien, C_s
9. Response modification factor, R
10. Analysis procedure used

Table 3.9: Occupancy category of buildings and other structures

Occupancy category	Nature of occupancy of buildings and other structures
I	Buildings and other structures that present low hazard to human life in the event of failure, e.g. agricultural facilities, certain temporary facilities, minor storage facilities, etc.
II	Buildings and other structures except those listed in occupancy category I, III and IV
III	Buildings and other structures that present substantial hazard to human life in the event of failure, i.e. school, jails and detention facilities, etc.
IV	in the event of failure designated as essential facilities, i.e. hospitals, fire and police stations, shelters, power-generating stations, structures containing highly toxic materials, aviation control towers, critical national defense, facilities required to maintain water pressure for fire suppression, etc.

2) Serviceability

Structural systems and members thereof shall be designed to have adequate stiffness to limit deflections and lateral drift. See ASCE 7 for drift limits applicable to earthquake loading.

3) Occupancy category

Buildinga shall be assigned an occupancy category in accordance with Table 3.9.

4) Load combinations†

For strength design or load and resistant factor design, the following load combinations are used.

$$\begin{cases} 1.2D + 1.0E + f_1 L + f_2 S \\ 0.9D + 1.0E + 1.6H \end{cases} \tag{3.42}$$

where, D is the dead load, E is the combined effect of horizontal and vertical earthquake induced forces, L is the live load, except roof live load, including any permitted live load reduction, S is the snow load, and H is the load due to lateral earthpressure, ground water pressure or pressure of bulk materials. f_1 is 1 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79kN/m^2), and 0.5 for other live loads. f_2 is 0.7 for roof configuration that do not shed snow off the structure, and 0.2 for other roof configuration.

For allowable stress design (working stress design), the following load combinations are used.

$$\begin{cases} D + H + F + (W \text{ or } 0.7E) \\ D + H + F + 0.75(W \text{ or } 0.7E) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R) \\ 0.6D + 0.7E + H \end{cases} \tag{3.43}$$

†Combinations that do not include the earthquake load are not shown in this summary.

where, F is the load due to fluids, W is the load due to wind pressure, L_r is the roof live load, and R is the rain load. In lieu of the above combinations, the following combination is permitted.

$$\begin{cases} D + L + S + E/1.4 \\ 0.9D + E/1.4 \end{cases} \tag{3.44}$$

5) Earthquake loads

i) Scope

Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resisit the effects of earthquake motions in accordance with ASCE 7.

ii) Mapped acceleration parameters

The parameters S_s and S_1 shall be determined from 0.2 (s) and 1 (s) spectral response acceleration shown in the figures[†]. Where $S_1 \leqq 0.04$ and $S_\mathrm{s} \leqq 0.15$, the structure is permitted to be assigned Seismic Design Category A.

iii) Site class

Base on the site soil properties, the site shall be classified as either Site Class A, B, C, D, E or F in accordance with Table 3.10.

iv) Site coefficients and adjusted maximum considered earthquake spectral response accleration

The maximum considered earthquake spectral response acceleration for short periods, S_MS, and 1(s) period, S_M1, adjusted for site class (see Table 3.10) shall be determined by:

$$\begin{cases} S_\mathrm{MS} = F_\mathrm{a}\, S_\mathrm{s} \\ S_\mathrm{M1} = F_\mathrm{v}\, S_1 \end{cases} \tag{3.45}$$

where, F_a is site coefficient defined in Table 3.11, and F_v is site coefficient defined in Table 3.12.

"Maximum Considered Earthquake (MCE)"

The maximum considered earthquake of IBC is based on a 2 % in 50 years (or return period of 2,500 years) ground motion. The response spectral accelerations in lieu of parameters tied to ground acceleration are shown in MCE ground motion maps, which are not shown in this summary. The resopnse spectral accelerations for MCE are $S_\mathrm{s} = 2.5$ (g) and $S_1 = 1.0$ (g) around Los Angels, California and $S_\mathrm{s} = 1.5$ (g) and $S_1 = 0.6$ (g) around San Francisco, California. The maximum value of S_s is in Saint Luis, Misurie, where $S_\mathrm{s} = 3.0$ (g) and $S_1 = 0.6$ (g). The values are very low at eastern part of U.S.A and the response values are less than 1/10 of the western coast of high seismicity area.

[†]The figures are not shown in this summary. See the commentary on "Maximum Considered Earthquake (MCE)"on p.87.

Table 3.10: Site class definitions

Site class	Soil profile name	Average properties in top 100 ft		
		Soil shear wave velocity $\bar{V}_s$ (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained strength $\bar{S}_u$ (psf)
A	Hard rock	$\bar{V}_s > 5000$	N/A	N/A
B	Rock	$2500 < \bar{V}_s \leqq 5000$	N/A	N/A
C	Very dense soil and soft rock	$1200 < \bar{V}_s \leqq 2500$	$\bar{N} > 50$	$\bar{S}_u \geqq 2000$
D	Stiff soil profile	$600 < \bar{V}_s \leqq 1200$	$15 \leqq \bar{N} \leqq 50$	$1000 \leqq \bar{S}_u \leqq 2000$
E	Soft soil profile	$\bar{V}_s < 600$	$\bar{N} < 15$	$\bar{S}_u < 1000$
E	—	Any profile with more than 10 ft of soil having the following characteristics: 1. Plasticity index PI > 20, 2. Moisture content $w \geqq 40$, and 3. Undrained shear strength $\bar{S}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soil vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly senstive clays, collapsible weakly cemented soils. 2. Peats and/or organic clays ($H > 10$ ft of peat and/or highly organic clay where $H =$ thickness of soils) 3. Very high plasticity clays ($H > 10$ ft with plasticity index PI > 75) 4. Very thick soft/medium stiff clays ($H > 120$ ft)		

1 ft = 304.8 mm, 1 ft^2 = 0.0929 m^2, 1 psf= 0.0479 kPa, N/A=Not applicable

v) Design spectral response acceleration parameters

5 % damped design spectral response acceleration at short period, S_{DS}, and at 1 (s) period, S_{D1}, shall be determined from:

$$\begin{cases} S_{DS} = \dfrac{2}{3} S_{MS} \\ S_{D1} = \dfrac{2}{3} S_{M1} \end{cases} \tag{3.46}$$

vi) Site classification for seismic design

Site classification for Site Class C, D or E shall be determined from Table 3.13.

The notation presented below apply upper 100 ft (30.48 m) of the soil porofile. Profiles containing different soil and/or rock layers shall be subdivided into those layers.

$$\bar{V}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \dfrac{d_i}{V_{si}}} \tag{3.47}$$

Table 3.11: Values of site coefficient F_a

Site class	Mapped spectral response acceleration at short period				
	$S_a \leqq 0.25$	$S_a = 0.50$	$S_a = 0.75$	$S_a = 1.00$	$S_a \geqq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	Values shall be determined in accordance with ASCE 7.				

Table 3.12: Values of site coefficient F_v

Site class	Mapped spectral response acceleration at 1 (s) period				
	$S_1 \leqq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geqq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	Values shall be determined in accordance with ASCE 7.				

where, V_{si} is the shear wave velocity, d_i is the thickness of any layer, and n is the number of soil layers in the upper 100 ft (30.48 m).

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \tag{3.48}$$

where, N_i is the Standard Penetration Resistance not exceeding 100 blows/ft.

$$\bar{N}_{\text{ch}} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \tag{3.49}$$

where, d_i and N_i are for cohesionless soil layers only, d_s is the total thickness of cohesionless soil layers, and m is the number of cohesionless soil layers.

$$\bar{S}_{\text{u}} = \frac{\sum_{i=1}^{k} d_i}{\sum_{i=1}^{k} \frac{d_i}{S_{\text{u}i}}} \tag{3.50}$$

where, $S_{\text{u}i}$ is the undrained shear strength, not to exceed 5,000 psf (240 kPa), d_c is the total thickness of cohesive soil layers, and k is the number of cohesive soil layers.

Where a site does not qualify under the criteria for Site Class F and there is a total thickness of soft clay greater than 10 ft (30.48 m) where a soft clay layer is defined by: $S_{\text{u}i} < 500$ psf (24 kPa), $w \geqq 40\,\%$, and PI > 20, it shall be classfied as Site Class E.

Table 3.13: Site classification

Site class	Shear wave velocity $\bar{V}_s$ (ft/s)	Standard penetration resistance $\bar{N}$ or $\bar{N}_{ch}$	Undrained shear strength $\bar{S}_u$ (psf)
E	$\bar{V}_s < 600$	$\bar{N} < 15$	$\bar{S}_u < 1000$
D	$600 < \bar{V}_s \leqq 1200$	$15 \leqq \bar{N} \leqq 50$	$1000 \leqq \bar{S}_u \leqq 2000$
C	$1200 < \bar{V}_s \leqq 2500$	$\bar{N} > 50$	$\bar{S}_u \geqq 2000$

1 ft=304.8 mm, 1 ft^2=0.0929 m^2, 1 psf=0.0479 kN/m^2

If the $\bar{S}_u$ is used and $\bar{N}_{ch}$ and $\bar{S}_u$ criteria differ, select the category with the softer soils.

vii) Seismic design category

Occupancy Category I, II or III structures located where the mapped spectral response acceleration parameter at 1 (s) period is $S_1 \geqq 0.75$ shall be assigned to Seismic Design Category E. Occupancy Category IV structures located where the mapped spectral response acceleration parameter at 1 (s) period is $S_1 \geqq 0.75$ shall be assigned to Seismic Design Category F. All other structures shall be assigned to a seismic design category based on their occupancy category and design spectral response acceleration coefficients, S_{DS} and S_{D1}, or the site-specific procedures of ASCE 7. Each building and structure shall be assigned to the more severe seismic deign category in acccordance with Table 3.14 or Table 3.15, irrespective of the fundamental period of vibration of the structure, T.

Table 3.14: Seismic design category based on S_{DS}

Value of S_{DS}	Occupancy category		
	I or II	III	IV
$S_{DS} < 0.167\,g$	A	A	A
$0.167\,g \leqq S_{DS} < 0.33\,g$	B	B	C
$0.33\,g \leqq S_{DS} < 0.50\,g$	C	C	D
$0.50 \leqq S_{DS}$	D	D	D

Table 3.15: Seismic design category based on S_{D1}

Value of S_{D1}	Occupancy category		
	I or II	III	IV
$S_{D1} < 0.067\,g$	A	A	A
$0.067\,g \leqq S_{D1} < 0.133\,g$	B	B	C
$0.133\,g \leqq S_{D1} < 0.20\,g$	C	C	D
$0.20 \leqq S_{D1}$	D	D	D

3.4 Seismic Design Criteria and Requirements of ASCE 7

ASCE 7[8] “Minimum Design Loads for Buildings and Other Structures” has 388 pages which includes 23 chapters, appendixes and commentary as follows:

Chapter 1 General
Chapter 2 Combination of Loads
Chapter 3 Dead Loads, Soil Loads, and Hydrostatic Pressure
Chapter 4 Live Loads
Chapter 5 Flood Loads
Chapter 6 Wind Loads
Chapter 7 Snow Loads
Chapter 8 Rain Loads
Chapter 9 Reserved for Future Provisions
Chapter 10 Ice Loads – Atmospheric Icing
Chapter 11 **Seismic Design Criteria**
Chapter 12 **Seismic Design Requirements for Building Structures**
Chapter 13 Seismic Design Requirements for Nonstructural Components
Chapter 14 Material - Specific Seismic Design and Detailing Requirements
Chapter 15 Seismic Design Requirements for Nonbuilding Structures
Chapter 16 Seismic Response History Procedures
Chapter 17 Seismic Design Requirements for Seismically Isolated Structures
Chapter 18 Seismic Design Requirements for Structures with Damping Systems
Chapter 19 Soil Structure Interaction for Seismic Design
Chapter 20 Soil Classification Procedure for Seismic Design
Chapter 21 Site-Specific Ground Motion Procedures for Seismic Design
Chapter 22 Seismic Ground Motion and Long-Period Transition Maps
Chapter 23 Seismic Design Reference Documents
(plus Appendixes and Commentary)

Chapter 11 “Seismic Design Criteria” and Chapter 12 “Seismic Design Requirements for Building Structures” of ASCE 7 are summarized as follows.

1) Seismic Ground Motion Values

i) Design response spectrum

The design response spectrum curve shall be developed as follows: (Where a MCE[†] response spectrum is required, it shall be determined by multiplying the design response spectrum

[†]Maximum Considered Earthquake: the most severe earthquake effects considered by this standard.

by 1.5.)

$$S_a = \begin{cases} S_{DS}\left(0.4 + 0.6\dfrac{T}{T_0}\right) & \text{for } T < T_0 \\ S_{DS} & \text{for } T_0 \leqq T \leqq T_S \\ \dfrac{S_{D1}}{T} & \text{for } T_S < T \leqq T_L \\ \dfrac{S_{DS}\,T_L}{T^2} & \text{for } T > T_L \end{cases} \tag{3.51}$$

Where, $T_0 = 0.2\,S_{D1}/S_{DS}$, $T_S = S_{D1}/S_{DS}$, T_L is the long-period transition periods (s).†

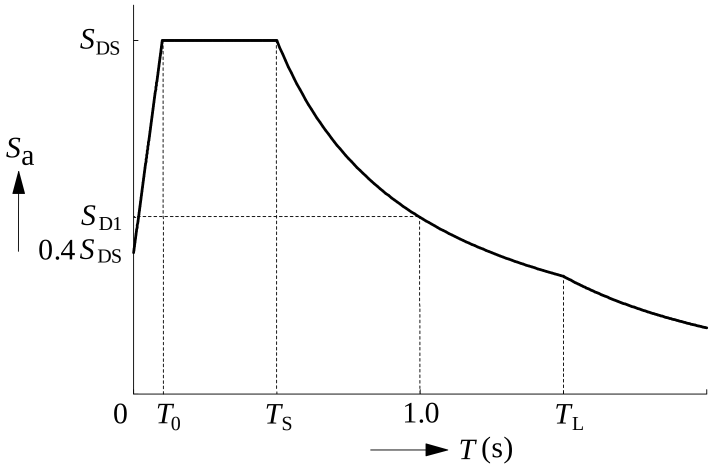


Figure 3.8: Design response spectrum
(MCE response spectrum is determined by multiplying 1.5.)

ii) Site-specific ground motion procedures

The site-specific ground motion procedures set force in Chapter 21‡ are permitted to be used to determine ground motions for any structure.

2) Importance Factor and Occupancy Category

An importance factor, I, shall be assigned to each structure in accordance with Table 3.16 based on the occupancy category.

3) Seismic Design Category

Occupancy Category I, II, or III structures located where the mapped spectral response acceleration parameter $S_1 \geqq 0.75$ shall be assigned to Seismic Design Category E. Occupancy Category IV structures located where the mapped spectral response acceleration parameter $S_1 \geqq 0.75$ shall

†U.S.A. are subdivided into 5 zones in terms of T_L, i.e. $T_L = 4, 6, 8, 12$, or 16 (s). T_L is the shortest of 0.4 (s) in the central part, and the longest of 16 (s) in the western coast and in Alaska.

‡Chapter 21 is not included in this summary.

Table 3.16: Important factors I

Ocuppancy category	Importance factor I
I or II	1.0
III	1.25
IV	1.5

be assigned to Seismic Design Category F. All other structures shall be assigned to a Seismic Design Category based on their Occupancy Category and the design spectral response acceleration parameters, S_{DS} and S_{D1}. Each building and structure shall be assigned to a more severe Seismic Design Category in accordance with Table 3.14† or Table 3.15.

4) Design Requirements for Seismic Design Category A

For purpose of analysis, the force at each level, F_x, shall be determined as follows:

$$F_x = 0.01\ w_x \tag{3.52}$$

where, w_x is the portion of the total dead load, D, located or assigned to Level x.

Any smaller portion of the structure shall be tied to the remainder of the structure with elements having design strength of not less than 5% of the portion's weight.

5) Structural System Selection

The basic lateral and vertical seismic force-resisting system shall conform to one of the types indicated in Table 3.17. The appropriate response modification coefficient R, system overstrength factor Ω_0, and the deflection amplification factor C_d, indicated in the table shall be used in determining the base shear, element design forces, and design story drift.

6) Irregular and Regular Classification

Structures having one or more of the irregularity types listed in Table 3.18 shall be designated as having horizontal structural irregularity. Structures having one or more of the irregularity types listed in Table 3.19 shall be designated as having vertical structural irregularity‡.

For structures assigned to Seismic Design Category D, E or F, ρ shall be equal to 1.3.

7) Seismic Load Effects and Combinations

i) Seismic load effect

The seismic load effect, E, shall be determined as follows:

†For short period structures, the Seismic Design Category is permitted to be determined to from Table 3.14 alone with some restrictions.

‡Structures having structural irregularities have some limitations for Seismic Design Categories and height, and those limitations are not included in this summary.

Table 3.17: Design coefficients and factors for seismic force-resisting systems*

				Structural system limitations and building height limit				
				Seismic design category				
Seismic force-resisting systems	R	Ω_0	C_d	B	C	D	E	F
Bearing wall systems								
Special reinforced concrete shear walls	5	2.5	5	NL	NL	160 ft	160 ft	100 ft
Ordinary reinforced concrete shear walls	4	2.5	4	NL	NL	NP	NP	NP
Moment-resisting frame systems								
Special steel moment frames	8	3	5.5	NL	NL	NL	NL	NL
Ordinary steel moment frames	3.5	3	3	NL	NL	NP	NP	NP
Dual systems with								
Special moment frames	8	2.5	4	NL	NL	NL	NL	NL
Intermediate moment frames	6	2.5	5	NL	NL	NP	NP	NP

*: More than 80 systems are included in the original Table.
R: Response modification coefficient, Ω_0: System overstrength factor,
C_d: Deflection amplification factor, NL: Not limited, NP: Not permitted

1.
$$E = E_h + E_v \tag{3.53}$$

where, E_h is the effect of horizontal seismic forces, and E_v is the effect of vertical.

The above E is for use in load combination as follows:

(Strength design)

$$1.2D + 1.0E + L + 0.2S \tag{3.54}$$

(Allowable stress design)

$$\begin{cases} D + H + F + (W \text{ or } 0.7E) \\ D + H + F + 0.75\,(W \text{ or } 0.7E) + 0.75L + 0.75\,(L_r \text{ or } S \text{ or } R) \end{cases} \tag{3.55}$$

2.
$$E = E_h - E_v \tag{3.56}$$

The above E is for use in load combination as follows:

(Strength Design)

$$0.9D + 1.0E + 1.6H \tag{3.57}$$

(Allowable Stress Design)

$$0.6D + 0.7E + H \tag{3.58}$$

(Horizontal seismic load effect) The horizontal seismic load effect, E_h, shall be determined as follows:

$$E_h = \rho\, Q_E \tag{3.59}$$

where, Q_E is effects of horizontal seismic forces from V or F_p, and ρ is the redundancy factor.

Table 3.18: Horizontal structural irregularities

	Irregularity type and description
1a.	**Torsional irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at the end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the ends of the structure.
1b.	**Extreme torsional irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at the end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the ends of the structure.
2.	**Reentrant corner irregularity** is defined to exist where both plan projection of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.
3.	**Diaphragm discontinuity irregularity** is defined to exist where there are diaphragms with abrupt discontinuities or variation in stiffness, including those having cutout or opening areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.
4.	**Out-of-plane offsets irregulariry** is defined to exist where there are discontinuities in lateral force-resistance path, such as out-of-plane offsets of the vertical elements.
5.	**Nonparallel system-irregularity** is defined to exist where the vertical lateral force-resisting elements are not parallel or symmetric about the major orthogonal axes of the seismic force-resisting system.

(Vertical seismic load effect) Vertical seismic load effect, E_v, shall be determined as follows:

$$E_v = 0.2\, S_{DS}\, D \tag{3.60}$$

where, S_{DS} is the design spectral response acceleration parameter at short period, and D is the effect of dead load.

ii) Seismic load combination[†]

Basic combinations for strength design are as follows:

$$\begin{cases} (1.2 + 0.2\, S_{DS})\, D + \rho\, Q_E + L + 0.2\, S \\ (0.9 - 0.2\, S_{DS})\, D + \rho\, Q_E + 1.6\, H \end{cases} \tag{3.61}$$

where, D is the effect of dead load, Q_E is the effect of horizontal seismic forces, L is the effect of live load, S is the effect of snow load, and H is the effect of soil load and hydrostatic pressure.

Basic combinations for allowable stress design are as follows:

$$\begin{cases} (1.0 + 0.14\, S_{DS})\, D + H + F + 0.7\, \rho\, Q_E \\ (1.0 + 0.105\, S_{DS})\, D + H + F + 0.525\, \rho\, Q_E + 0.75\, L + 0.75\, (L_r \;\text{ or }\; S \;\text{ or }\; R) \\ (0.6 - 0.14\, S_{DS})\, D + 0.7\, \rho\, Q_E + H \end{cases} \tag{3.62}$$

[†]Combinations that do not include the seismic load are not shown in this summary.

Table 3.19: Vertical structural irregularities

	Irregularity type and description
1a.	**Stiffness-soft story irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 70 % of that in the story above or less than 80 % of the average stiffness of the three stories above.
1b.	**Stiffness-extreme soft story irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 60 % of that in the story above or less than 70 % of the average stiffness of the three stories above.
2.	**Weight (mass) irregularity** is defined to exist where the effective mass of any story is more than 150 % of the effective mass of an adjacent story.
3.	**Vertical geometric irregularity** is defined to exist where the horizontal dimension of the seismic force-resisting system in any story is more than 130 % of that in an adjacent story.
4.	**In-plane discontinuity in vertical lateral force-resisting element irregularity** is defined to exist where an in-plane offset of the lateral force-resisting elements is greater than the length of those elements or there exists a reduction in stiffness of the resisting element in the story below.
5a.	**Discontinuity in lateral strength-weak story irregularity** is defined to exist where the story lateral strength is less than 80 % of that in the story above.
5b.	**Discontinuity in lateral strength-extreme weak story irregularity** is defined to exist where the story lateral strength is less than 65 % of that in the story above.

where, F is the effect of load due to fluid, L_r is the effect of roof live load, and R is the effect of rain load.

iii) Seismic load effect including overstrength factor

Where spacifically required, conditions requiring overstrength factor applications shall be determined as follows†:

$$E_{\mathrm{m}} = E_{\mathrm{mh}} \pm E_{\mathrm{v}} \tag{3.63}$$

where, E_{mh} is the effect of horizontal seismic forces including structural overstrength.

The horizontal seismic load effect with overstrength factor, E_{mh}, shall be determined as follows:

$$E_{\mathrm{mh}} = \Omega_0\, Q_{\mathrm{E}} \tag{3.64}$$

where, Ω_0 is the overstrength factor.

iv) Load combinations with overstrength factor

Basic combinations for strength design with overstrength factor are as follows:

$$\begin{cases} (1.2 + 0.2\, S_{\mathrm{DS}})\, D + \Omega_0\, Q_{\mathrm{E}} + L + 0.2\, S \\ (0.9 - 0.2\, S_{\mathrm{DS}})\, D + \Omega_0\, Q_{\mathrm{E}} + 1.6\, H \end{cases} \tag{3.65}$$

†Load combinations applied are the same as Eqs.(3.53) and (3.56).

Basic combinations for allowable stress design with overstrength factor are as follows:

$$\begin{cases} (1.0 + 0.14\, S_{\mathrm{DS}})\, D + H + F + 0.7\, \Omega_0\, Q_{\mathrm{E}} \\ (1.0 + 0.105\, S_{\mathrm{DS}})\, D + H + F + 0.525\, \Omega_0\, Q_{\mathrm{E}} + 0.75\, L + 0.75\, (L_r \ \text{ or } \ S \ \text{ or } \ R) \\ (0.6 - 0.14\, S_{\mathrm{DS}})\, D + 0.7\, \Omega_0\, Q_{\mathrm{E}} + H \end{cases} \tag{3.66}$$

Allowable stresses are permitted to be determined using an allowable stress increase of 1.2.

v) Minimum upward force for horizontal cantilevers

In structures assigned to Seismic Design Category D, E or F, horizontal cantilever structural components shall be designed for a minimum net upward force of 0.2 times the dead load in addition to the applicable load combinations.

8) Direction of Loading

The directions of application of seismic forces used in the design shall be those which will produce the most critical load effects. It is permitted to satisfy this requirement using the procedure as below.

- For structures assigned to Seismic Design Category B, the design seismic forces are permitted to be applied independently in each of two orthogonal directions and orthogonal interaction effects are permitted to be neglected.
- Loading applied to structures assigned to Seismic Design Category C shall, as a minimum, conform to the requirements for Seismic Design Category B. Structures that have horizontal structural irregularity Type 5 in Table 3.18 shall use one of the following procedures:
  a) components and foundations are designed for the load combination: 100 % of the forces for one direction plus 30 % of the forces for the perpendicular direction,
  b) the linear or nonlinear response history procedure with orthogonal pairs of ground motion acceleration histories applied simultaneously.
- Structures assigned to Seismic Design Category D, E or F, shall, as a minimum, conform to the requirements for Seismic Design Category C. In addition, any column and wall that forms part of two or more intersecting seismic force-resisting systems and is subjected to axial load due to seismic forces eqaul to or exceeding 20 % of the axial design strength of the column or wall shall be designed for the most critical load effect due to application of seismic forces in any direction. Either of the above procedures a) or b) are permitted to be used to satisfy this requirements.

9) Analysis Procedure Selection

The structural analysis required shall consist of one of the types permitted in Table 3.20.

Table 3.20: Permitted analytical procedures

Seismic design category	Structural characteristics	(a)	(b)	(c)
B, C	Occupncy Category I or II buildings of light-framed construction not exceeding 3 stories in height	P	P	P
	Other Occupncy Category I or II buildings not exceeding 3 stories in height	P	P	P
	All other structures	P	P	P
D, E, F	Occupncy Category I or II buildings of light-framed construction not exceeding 3 stories in height	P	P	P
	Other Occupncy Category I or II buildings not exceeding 3 stories in height	P	P	P
	Regular structures with $T < 3.5T_s$ and all structures of light frame construction	P	P	P
	Irregular structures with $T < 3.5T_s$ and having only horizontal irregularities Type 2, 3, 4 or 5 of Table 3.18 or vertical irregularities Type 4, 5a or 5b of Table 3.19.	P	P	P
	All other structures	NP	P	P

(a) Equivalent lateral force analysis, (b) Modal response spectrum analysis,
(c) Seismic response history procedure, P: Permitted, NP: Not permitted

10) Modeling Criteria

i) Foundation modeling

It is permitted to consider the structure to be fixed at the base†.

ii) Effective seismic weight

The effective seismic weignt, W, of a structure shall include the dead load and other loads as listed below:

- In areas used for garage, a minimum of 25 % of floor live load (floor live load in public garages and open parking structures need not be included).
- Where provisions for partitions is required in the floor load design, the actual partition weight or a minimum weight of 10 psf (0.48 kN/m^2) of floor area, whichever is greater.
- Total operating weight of permanent equipment.
- Where the flat roof snow load, P_f, exceeds 30 psf (1.44 kN/m^2), 20 % of the uniform design snow load, regardless of actual roof slope.

iii) Structural modeling

- A mathematical model of the structure shall be constructed for the purpose of determining member forces and structure displacements resulting from applied loads and any imposed displacements or P-Δ effects.

† It is also permitted to consider the foundation flexibility.

- Structures that have horizontal structural irregularity Type 1a, 1b, 4 or 5 of Table 3.18 shall be analyzed using 3D representation.
- Where the diaphragms have not been classified as rigid or flexible, the model shall include representation of the diaphragm's stiffness characteristics.
- Stiffness properties of concrete and masonry elements shall consider the effects of cracked section. For steel moment frame systems, the contribution of panel zone deformation to overall story drift shall be included.

iv) Interaction effects

Moment-resisting frames that are enclosed or adjoined by elements that are more rigid and not considered to be part of the seismic force-resisting system shall be designed so that the action or failure of those elements will not impair the vertical load and seismic force-resisting capability of the frame. The design shall provide for the effect of these rigid elements on the structural system at structural deformation corresponding to the story drift. In addition, the effects of these elements shall be considered where determining whether a structure has one or more of the irregularities.

11) Equivalent Lateral Force Procedure

i) Seismic Base Shear

The seismic base shear, V, in a given direction shall be determined as follows:

$$V = C_s\, W \tag{3.67}$$

where, C_s is the seismic response coefficient[†], and W is the effective seismic weight.

The seismic response coefficient, C_s, shall be determined as follows:

$$C_s = \frac{S_{DS}}{(R/I)} \tag{3.68}$$

where, S_{DS} is the design spectral response acceleration parameter in the short period range, R is the response modification factor in Table 3.17, and I is the occupancy importance factor.

The value of C_s shall be as follows:

$$C_s \leqq \begin{cases} \dfrac{S_{D1}}{T(R/I)} & \text{for} \quad T \leqq T_L \\ \dfrac{S_{D1}\, T_L}{T^2(R/I)} & \text{for} \quad T \geqq T_L \end{cases} \tag{3.69}$$

$$C_s \geqq 0.01 \tag{3.70}$$

In addition, for structures located where $S_{D1} \geqq 0.6$g,

$$C_s \geqq \frac{0.5\, S_1}{(R/I)} \tag{3.71}$$

[†]It is usually called the "base shear coefficient".

Table 3.21: Coefficient for upper limit on calculated period

Design spectral response acceleration parameter at 1 (s), S_{D1}	Coefficient C_u
$\geqq 0.4$	1.4
0.3	1.4
0.2	1.5
0.15	1.6
$\leqq 0.1$	1.7

(Soil structure interaction reduction) A soil structure interaction reduction is permitted where determined using Chapter 19‡ or other generally accepted procedures.

(Maximum S_s value in determination of C_s) For regular structures five stories or less in height and having a period $T \leqq 0.5$ (s), C_s is permitted to be calculated using $S_s = 1.5$.

ii) Period Determination

The fundamental period of the structure, T, in the direction under consideration shall be established using the structural properties and deformation characteristics of the resisting elements in a properly substantiated analysis. The funamental period, T, shall not exceed the product of the coefficient for upper limit on calculated period, C_u, from Table 3.21 and the approximate fundamental period, T_a, determined from Eq.(3.72). As an alternative, it is permitted to use the apporximate building period, T_a (s), as follows:

$$T_a = C_t\, h_n^x \tag{3.72}$$

where, h_n (ft) is the height above the base to the highest level of the structure, and the coefficient C_t and x are determined from Table 3.22.

Alternately, it is permitted to determine the approximate fundamental period, T_a (s), from the following equation for structures not exceeding 12 stories in height in which the seismic force-resisting system consists entirely of concrete or steel moment frames and the story height is at least 10 (ft) or 3 (m):

$$T_a = 0.1N \tag{3.73}$$

where, N is the number of stories.

The approximate fundamental period, T_a (s), for masonry or concrete shear wall structures is permitted to be detrmined as follows:

$$T_a = \frac{0.0019}{\sqrt{C_w}} h_n \tag{3.74}$$

where, C_w is calculated as follows:

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]} \tag{3.75}$$

‡Chapter 19 is not included in this summary.

Table 3.22: Values of approximate period parameters C_t and x

Structural type	C_t	x
Steel moment-resisting frames*	0.028 (0.0742)	0.8
Concrete moment-resisting frames*	0.016 (0.0466)	0.8
Eccentrically braced steel frames	0.03 (0.0731)	0.75
All other structural systems	0.02 (0.0488)	0.75

Metric equivalents are shown in parentheses.
* Moment-resisting frame systems in which the frames resist 100 % of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces.

where, A_B (ft^2) is the area of base of structure, A_i (ft^2) is the web area of shear wall "i", D_i (ft) is the length of shear wall "i", h_i (ft) is the height of shear wall "i", and x is the number of shear walls in the direction under consideration.

iii) Vertical distribution of seismic forces

The lateral seismic force, F_x (kip or kN), induced at any level shall be determined as follows:

$$F_x = C_{vx}\, V \tag{3.76}$$

where, V (kip or kN) is the total design lateral force or shear at the base of the structure, and C_{vx} is the vertical distribution factor determined as follows:

$$C_{vx} = \frac{w_x\, h_x^k}{\sum_{i=1}^{n} w_i\, h_i^k} \tag{3.77}$$

where, w_i and w_x are the portion of total effective seismic weight of the structure, W, located or assigned to Level i or x. h_i are h_x are the height (ft or m) from the base to Level i or x. k is an exponent related to the structure period as follows: for structures having a period of 0.5 (s) or less, $k = 1$, for structures having a period of 2.5 (s) or more, $k = 2$, and for structures having a period of between 0.5 (s) and 2.5 (s), k shall be 2 or shall be determined by linear interpolation between 1 and 2.

iv) Horizontal distribution of forces

The seismic desigh story shear in any story, V_x (kip or kN), shall be determined as follows:

$$V_x = \sum_{i=x}^{n} F_i \tag{3.78}$$

where, F_i (kip or kN) is the portion of the seismic base shear V_x (kip or kN) induced at Level i.

The seismic design story shear V_x (kip or kN) shall be distributed to the various vertical elements of the seismic force-resisting system in the story under consideration based on the relative lateral stiffness of the vertical resisting elements and the diaphragm†.

†Inherent torsion, accidental torsion and amplification of accidental torsional moment are not included in this summary.

v) Overturning

The structure shall be designed to resist overturning effects caused by the seismic forces.

vi) Story drift determination

The design story drift Δ shall be computed as the difference of the deflections at the center of mass at the top and bottom of the story under consideration. Where allowable stress design is used, Δ shall be computed using the strength level seismic forces without reduction for allowable stress design.

The deflection of Level x at the center of mass δ_x shall be determined as follows:

$$\delta_x = \frac{C_\mathrm{d}\,\delta_{x\mathrm{e}}}{I} \tag{3.79}$$

where, C_d is the deflection amplification factor in Table 3.17, $\delta_{x\mathrm{e}}$ is the deflection determined by an elastic analysis, and I is the importance factor.

(Minimum base shear for computing drift) The elastic analysis of the seismic force-resisting system shall be made using the prescribed design forces of the equivalent lateral force procedure.

(Period for computing drift) For determining compliance with the story drift limits, it is permitted to determine the elastic drifts $\delta_{x\mathrm{e}}$, using seismic design forces based on the computed fundamental period of the structure without the upper limit $C_\mathrm{u}T_\mathrm{u}$.

vii) P-Δ effects

P-Δ effects on the story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered where stability coefficient θ as determined by the following equation is equal to or less than 0.10:

$$\theta = \frac{P_x\,\Delta}{V_x\,h_{sx}\,C_\mathrm{d}} \tag{3.80}$$

where, P_x (kip or kN) is the total vertical design load at and above Level x, where computing P_x, no individual load factor need exceed 1.0, Δ (in. or mm) is the design story drift occurring simultaneously with V_x, V_x (kip or kN) is the seismic shear force acting between Level x and $x-1$, h_{sx} (in. or mm) is the story height below Level x, and C_d is the deflection amplification factor in Table 3.17.

The stability coefficient θ shall not exceed θ_max determined as follows:

$$\theta_\mathrm{max} = \frac{0.5}{\beta\,C_\mathrm{d}} \leqq 0.25 \tag{3.81}$$

where, β is the ratio of shear demand to shear capacity for the story Levels x and $x-1$. The ratio is permitted to be conservatively taken as 1.0.

Where the stability coefficient θ is $0.10 < \theta \leqq \theta_\mathrm{max}$, the incremental factor related to P-Δ effects on displacements and member forces shall be determined by rational analysis. Alternately, it is permitted to multiply displacements and member forces by $1.0/(1-\theta)$.

Where $\theta > \theta_\mathrm{max}$, the structure is potentially unstable and shall be redesigned.

Where P-Δ effect is included in an automated analysis, Eq.(3.81) shall still be satisfied, however, the value of θ computed from Eq.(3.80) using the results of the P-Δ analysis is permitted to be divided by $(1+\theta)$ before checking Eq.(3.81).

12) Modal Response Spectrum Analysis

i) Number of modes

An analysis shall be conducted to determine the natural modes of vibration for the structure. The analysis shall include a suifficient number of modes to obtain a combined modal mass paticipation of at least 90 % of the actural mass in each of the orthogonal horizontal directions of response considered by the model.

ii) Modal response parameters

The value for each force-related design parameter of interest, including story drifts, support forces and individual member forces for each mode of response shall be computed using the properies of each mode and the response spectra defined on p.92 or Section 21.2[†] divided by (R/I). The value for displacement and drift quantities shall be multiplied by (C_d/I).

iii) Combined response parameters

The value for each parameter of interest calculated for various modes shall be combined using either the square root of the sum of the squares method (SRSS) or the complete quadratic combination method (CQC). The CQC method shall be used for each of modal values or where closely spaced modes that have significant cross-correlation of translational and torsional response.

iv) A scaling design values of combined response

A base shear V shall be calculated in each of the two orthogonal horizontal directions using the calculated fundamental period of the structure T in each direction, except where the calculated funamental period exceeds $C_\mathrm{u}T_\mathrm{a}$, then $C_\mathrm{u}T_\mathrm{a}$ shall be used in lieu of T in that direction. Where the combined response for the modal base shear V_t is less than 85% of the calculated base shear V using the equivalent lateral force procedure, the forces, but not drifts, shall be multiplied by $0.85V/V_\mathrm{t}$.

v) Horizontal shear distribution

The distribution of horizontal shear shall be in accordance with the requirements in the equivalent lateral force procedures, except that amplification of torsion is not required where accidental torsional effects are included in the dynamic analysis model.

vi) P-Δ effects

P-Δ shall be determined as the equivalent lateral force procedure.

vii) Soil struture interaction reduction

A soil structure interaction reduction is permitted where determined using Chapter 19[‡] or other generally accepted procedures.

[†]Section 21.2 is not included in this summary.
[‡]Chapter 19 is not included in this summary.

13) Drift and deformation

i) Story drift limit

The design story drift Δ shall not exceed the allowable story drift Δ_a as obtained from Table 3.23 for any story. For structures with significant tosional deflections, the maximum drift shall include torsional effects. For structures assigned to Seismic Design Category C, D, E or F having horizontal irregularity Types 1a or 1b of Table 3.18, the design story drift Δ shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the story under consideration.

Table 3.23: Allowable story drift*, Δ_a

Structure	Occupancy category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall system that have been designed to accomodate the story drifts.	0.025 h_{sx}	0.020 h_{sx}	0.015 h_{sx}
Masonry cantilever shear wall structures**	0.010 h_{sx}	0.010 h_{sx}	0.010 h_{sx}
Other masonry shear wall structures	0.007 h_{sx}	0.007 h_{sx}	0.007 h_{sx}
All other structures	0.020 h_{sx}	0.015 h_{sx}	0.010 h_{sx}

h_{sx} is the story height below Level x.

For seismic force-resisting system comprised solely of moment frames in Seismic Design Categories D, E and F, the allowable story drift shall comply with the requirements of 13) ii).

* There shall be no drift limit for single-story structures with interior walls, partitions, ceilings and exterior wall systems that have been designed to accomodate the story drifts. The structure separation requirement is not waived.

** Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

ii) Moment frames in structures assigned to seismic design categories D through F

For seismic force-resisting system comprised solely of moment frames in structures assigned to Seismic Design Categories D, E or F, the design story drift Δ shall not exceed Δ_a/ρ for any story.

iii) Diaphragm deflection

The deflection in the plane of the diaphragm, as determined by engineering analysis, shall not exceed the permissible deflection of the attached elements.

iv) Building separation

All portion of the structure shall be designed and constructed to act as an integral unit in resisting seismic forces unless separated structually by a distance sufficient to avoid damaging contact under total deflection δ_x.

v) Deformation compatibility for Seismic Design Category D through F

For structures assigned to Seismic Design Category D, E or F, every structural component not included in the seismic force-resisting system in the direction under consideration shall be designed to be adequate for the gravity load effects and seismic forces resulting from displacement to the design story drift Δ.

14) Simplified lateral force analysis procedure

i) Application

The structure shall be considered fixed at the base.

ii) Seismic base shear

The seismic base shear V in a given direction shall be determined as follows:

$$V = \frac{F\,S_{\mathrm{DS}}}{R} W \tag{3.82}$$

wnere, F is 1.0 for one-story buildings, 1.1 for two-story buildings and 1.2 for three-story buildings.

$$S_{\mathrm{DS}} = \frac{2}{3} F_{\mathrm{a}} S_{\mathrm{s}} \tag{3.83}$$

iii) Vertical distribution

The forces at level x, F_x, shall be calculated as follows:

$$F_x = \frac{w_x}{W} V \tag{3.84}$$

where, w_x is the portion of the effective seismic weight of the structure W at level x.

iv) Horizontal shear distribution

(Flexible diaphragm structure) The seismic design story shear in stories of structures with flexible diaphragms shall be distributed to the vertical elements of the lateral force resisting system using tributary area rules.

(Structures with diaphragm that are not flexible) For structures with diaphragms that are not flexible, the seismic design story shear shall be distributed to the various vertical elements of the seismic force-resisting system under consideration based on the relative lateral stiffness of the vertical elements and the diaphragm.

(Torsion) The design of structures with diaphragms that are not flexible shall include the torsional moment M_{t} resulting from eccentricity between the locations of center of mass and the center of rigidity.

v) Overturning

The structure shall be designed to resist overturning effects caused by the seismic forces. The foundation of structures shall be designed for not less than 75 % of the foundation overturning design moment M_{f} at the foundation-soil interaction.

vi) Drift limits and building separation

Structural drift need not be calculated. Where a drift value is needed for use in material standards, to determine structural separation between buildings, for design of claddings, or for other design requirements, it shall be taken as 1 % of building height unless computed to be less.

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