Invited Paper

Earthquake Resistant Design of Reinforced Concrete Buildings
Past and Future

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Abstract

This paper briefly reviews the development of earthquake resistant design of buildings. Measurement of ground acceleration started in the 1930s, and response calculation was made possible in the 1940s. Design response spectra were formulated in the late 1950s to 1960s. Non-linear response was introduced in seismic design in the 1960s and the capacity design concept was generally introduced in the 1970s for collapse safety. The damage statistics of reinforced concrete buildings in the 1995 Kobe disaster demonstrated the improvement of building performance with the development of design methodology. Buildings designed and constructed using out-dated methodology should be upgraded. Performance-based engineering should be emphasized, especially for the protection of building functions following frequent earthquakes.

1. Introduction

An earthquake, caused by a fault movement on the earth surface, results in severe ground shaking leading to the damage and collapse of buildings and civil-infra-structures, landslides in the case of loose slopes, and liquefaction of sandy soil. If an earthquake occurs under the sea, the associated water movement causes high tidal waves called tsunamis.

Earthquake disasters are not limited to structural damage and injury/death of people under collapsed structures. Fire is known to increase the extent of the disaster immediately after an earthquake. The breakage of water lines reduces fire fighting capability in urban areas. The affected people need support, such as medical treatment, food, clean water, accommodations and clothing. Continued service of lifeline systems, such as electricity, city gas, city water, communication lines and transportation, is essential for the life of affected people. Damage to highway viaducts or railway, as seen in the 1995 Kobe earthquake disaster, can delay evacuation and rescue operations. It is the responsibility of civil and building engineers to provide society with the technology to build safe environments.

Reinforced concrete has been used for building construction since the middle of the 19th century, first for some parts of buildings, and then for the entire building structure. Reinforced concrete is a major construction material for civil infrastructure in current society. Construction has always preceded the development of structural design methodology. Dramatic collapse of buildings has been observed after each disastrous earthquake, resulting in loss of life. Various types of damage have been identified through the investigation of damages. Each damage case has provided important information regarding the improvement of design and construction practices and attention has been directed to the prevention of structural collapse to protect the occupants of building in the last century.

Thank to the efforts of many pioneering researchers and engineers, the state of the art in earthquake resistant design and construction can reduce the life threat in reinforced concrete buildings. Attention should be directed to the protection of existing structures constructed using old technology. The vulnerability of these existing structures should be examined and seismically deficient structures should be retrofitted. One of the important research targets in present earthquake engineering is the development of design methodology to maintain building functions after infrequent earthquakes, for example, through the application of structural control technology.

This paper reviews the development of earthquake engineering in relation to earthquake resistance of buildings and discusses the current problems in earthquake engineering related to reinforced concrete construction.

2. Development of seismology and geophysics

Earthquake phenomena must have attracted the curiosity of scientists in the past. Ancient Greek sophists hypothesized different causes for earthquakes. Aristotle (383-322 B.C.), for example, related atmospheric events such as wind, thunder and lightning, and subterranean events, and explained that dry and smoky vapors caused earthquakes under the earth, and wind, thunder, lightning in the atmosphere. Aristotle's theory was believed through the Middle Ages in Europe. The 1755 Lisbon Earthquake (M8.7), which killed 70,000, partially due to a tsunami tidal wave, and a series of earthquakes in

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London in 1749 and 1750 attracted the interest of scientists.

The first scientific investigation about earthquake phenomena is believed to have been carried out by Robert Mallet, who initiated the physico-mechanical investigation of earthquake wave propagation. He investigated the earthquake phenomena of the 1857 Naples Earthquake, and used such technical terms as "seismology," "hypocenter," "isoseismal," and "wave path" in his report (Mallet, 1862).

The measurement of earthquake ground vibrations must have been a challenge for scientists. Chan Heng, in 132 A.D. in China, developed an instrument to detect earthquakes and point out the direction of the epicenter. Mallet also invented an instrument to record the intensity of ground motion by measuring the direction and distance of a particle moved by the motion. Many attempts were made to develop seismometers (seismographs) that could record ground movement during earthquakes. Luigi Palmieri developed an electromagnetic seismograph in 1855. One was installed near Mount Vesuvius, and another one at the University of Naples. The Ministry of the Interior of Japan adopted Palmieri-type seismometers in 1875.

The first seismological society in the world, the Seismological Society of Japan, was founded in 1880 when European and U.S. engineering professors, invited to the College of Engineering in Tokyo, were interested in the 1880 Yokohama earthquake (M5.5), which caused minor damage to buildings, but collapsed a chimney. John Milne, professor of Geology and Mining at the Engineering College, was the leader in scientific and engineering research. Milne, together with J. A. Ewing and T. Gray, developed a modern three-directional seismometer in 1881. Important research findings were published in the transactions. For example, Milne introduced Mallet's work on seismology and Ewing noted the difference between primary and secondary waves in the recorded ground motion.

The University of Tokyo was renamed as the Imperial University in 1886. Kiyokage Sekiya, who worked closely with Ewing and Milne, became the first professor of seismology chair at the Faculty of Science. Fusakichi Omori who succeeded him in 1897, was active in experimental as well as theoretical research for earthquake disaster mitigation.

The relation between fault movements and earthquakes was pointed out by Grove K. Gilbert, a U.S. geologist, who reported in 1872 that earthquakes usually centered around a fault line. Clear relative movement was observed across the San Andreas Fault after the 1906 San Francisco Earthquake (Ms 8.3). This earthquake caused 700 to 800 deaths and destroyed 28,188 buildings. The main source of disaster was fire. Harry F. Reid, Professor at Johns Hopkins University, presented the "Elastic Rebound Theory" in 1908 to describe the process of an earthquake mechanism; "... external forces must have produced an elastic strain in the region about the fault-line, and the stresses thus induced were the forces which caused the sudden displacements, or elastic rebounds, when the rupture occurred...." Reid did not explain what causes the external forces acting along fault lines.

Recent developments in geophysics are fascinating; especially research on the relationship between plate tectonics and earthquakes. Alfred Wegener presented the theory of continental drift (Wegener, 1915). He provided extensive supporting evidence for his theory such as geological formations, fossils, animals and climatology. He claimed that a single mass, called Pangaea, drifted and split to form the current continents. Wegener, however, had no convincing mechanism to explain the continental drift. Exploration data regarding the earth's crust, notably the ocean floor, increased in the 1950s; e.g., American physicists M. Ewing and B. Heezen discovered the great global rift (the Mid-Ocean Ridge in the Atlantic Ocean). On the basis of such exploration data, H. Hess, professor of Geology at Princeton University, proposed the theory of sea-floor spreading in 1960, which provided a mechanism to support Wegener's continental drift. Plate tectonics can describe the accumulation of strains at the boundaries of adjacent plates or within a plate due to plate movements at the earth's surface, which cause earthquakes.

Major earthquakes occur along the boundary of moving tectonic plates when the strain energy, accumulated by the resistance against inter-plate movement, is suddenly released. This type of inter-plate earthquakes occurs repeatedly at a relatively short interval of 50 to 200 years. Seismically blank regions, where seismic activity is quiet for some time along the tectonic plate boundary, are identified as the location of future earthquake occurrences. However, it is not possible at this stage to accurately predict the time, location and magnitude of earthquake occurrences.

Another type of earthquakes occurs within a tectonic plate when the stress accumulated within a plate by the pressure of peripheral plate movements, exceeds the resisting capacity of the rock layers at the fault. The epicenter is relatively shallow within 30 km from the earth surface. The fault in a plate remains as a weak spot after an earthquake, and earthquakes occur repeatedly at the same location if stress accumulates up to the failure level. The location of many active faults has been identified by geologists, and is taken into consideration in developing a seismicity map for structural design. If an intra-plate earthquake occurs near a city, the disaster in densely populated areas can be significant. It should be noted that this type of intra-plate earthquakes occurs at a long interval of 1,000 to 3,000 years. Therefore, it is more difficult to accurately predict the time, location and magnitude of intra-plate earthquakes.

We need to emphasize the need for disaster mitigation measures in society focusing on optimum use of seismology and geophysics data.

3. Dawn of earthquake engineering

It should be noted that Sir Isaac Newton, in 1687 proposed the law of motion in "Philosophia Naturalis Principia Mathematica"; i.e., when a force acts on a particle, the resultant acceleration of the particle is directly proportional to the force. The equation was introduced to calculate the motion of stars in the universe. The law of motion was introduced in engineering by J. R. d'Alembert who proposed the so-called D'Alembert's principle in his "Traité de Dynamique" in 1743; i.e., the equilibrium of forces can be discussed in a dynamic problem by introducing a fictitious inertia force, proportional to the acceleration and mass of a particle but acting in the direction opposite to the acceleration.

John William Strut, also known as Lord Rayleigh, in his "Theory of Sound" published in 1877, discussed the vibration of a single-degree-of-freedom system with viscous damping under harmonic excitation, longitudinal, torsional and lateral vibration of bars, and the vibration of membranes, plates and shells. Such knowledge could not be used in earthquake engineering for many years because the ground acceleration signal of an earthquake was not measured and because the equation of motion could not be solved for an arbitrary excitation function.

3.1 Intensity of ground motion

Early earthquake engineers and seismologists must have known the importance of ground acceleration to estimate inertia forces acting on structures during an earthquake. The seismograph, however, was not capable of measuring ground acceleration, which was more important for engineering purposes. E. S. Holden (1888), Director of the Lick Observatory in California, reported that "The researches of the Japanese seismologists have abundantly shown that the destruction of buildings, etc., is proportional to the acceleration produced by the earthquake shock itself in a mass connected with the earth's surface."

Indeed in Japan, efforts were made to estimate the maximum ground acceleration during an earthquake. John Milne and his student, Kiyokage Sekiya, estimated maximum ground acceleration amplitudes from the measured seismograph (displacement) records by assuming harmonic motions in 1884. Because the dominant frequencies in displacement and acceleration signals were different, this method tended to underestimate the maximum acceleration. Milne (1885) introduced the West's equation, which was used to estimate maximum ground acceleration α necessary to overturn a rigid body of width b and height h attached on the ground simply using dynamic equilibrium (**Fig. 1**);

$$\alpha > \frac{b}{h} \tag{1}$$

where acceleration α is expressed as the ratio to gravitational acceleration. This method was extensively used in Japan to estimate the intensity of ground motions from the dimensions of overturned tomb stones after an earthquake.

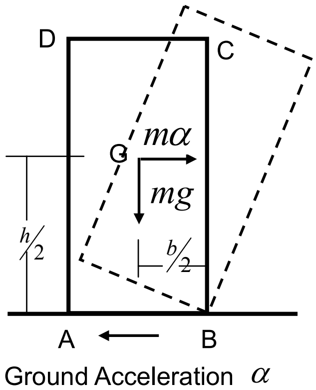

Fig. 1 The West's equation.

The 1891 Nohbi Earthquake (M 8.0) caused significant damage to then modern brick and masonry structures in Nagoya City. This is a largest-class near-field earthquake to have occurred in Japan. 7,273 were killed in sparsely populated areas, and 142,177 houses were destroyed. Milne and Burton (1891) recorded the disaster. Milne, after noting the effect of surface geology on the damage rate, pointed out that "we must construct, not simply to resist vertically applied stresses, but carefully consider effects due to movements applied more or less in horizontal directions." He could not define the intensity of lateral forces to be used in design. The Japanese Government established the Earthquake Disaster Prevention Investigation Council in 1892 for the promotion of research in earthquake engineering and seismology, and for implementing research findings in practice. The Seismological Society of Japan was merged into this council.

3.2 Seismic design forces

The first quantitative seismic design recommendations were made after the 1908 Messina Earthquake in Italy, which killed more than 83,000 people. Housner (1984) stated that "The government of Italy responded to the Messina earthquake by appointing a special committee composed of nine practicing engineers and five professors of engineering ... M. Panetti, Professor of Applied Mechanics in Turin ... recommended that the first story be designed for a horizontal force equal to 1/12 the weight above and the second and third stories to be designed for 1/8 of the building weight above." The height of buildings was limited to three stories. The technical background for this quantification is not clear, but it is interesting to note that design seismic forces were initially defined in terms of a story shear coefficient, a ratio of story shear to weight above, rather than a seismic coefficient, a ratio of the horizontal force of a floor to the weight of the floor.

Riki (Toshikata) Sano (1916) proposed the use of seismic coefficients in earthquake resistant building design. He assumed a building to be rigid and directly connected to the ground surface, and suggested a seismic coefficient equal to the maximum ground acceleration normalized to gravity acceleration G. Although he noted that lateral acceleration response might be amplified from the ground acceleration with lateral deformation of the structure, he ignored the effect in determining the seismic coefficient. He estimated the maximum ground acceleration in the Honjo and Fukagawa areas on alluvial soft soil in Tokyo to be 0.30 G and above on the basis of the damage to houses in the 1855 Ansei-Edo (Tokyo) Earthquake, and that in the Yamanote area on diluvial hard soil to be 0.15 G. Sano also discussed earthquake damage to brick masonry, steel, reinforced concrete and timber houses and buildings and proposed methods to improve the earthquake resistance of such structures.

3.3 Structural analysis methods

Building structures are highly statically indeterminate. Actions and stresses in a building must be calculated before seismic forces can be utilized in design. Fundamental studies of structures were developed in the middle of the nineteenth century. J. C. Maxwell in 1864 and O. Mohr in 1874 separately developed the unit load method to determine the deflection of elastic trusses and the flexibility method to determine redundant forces in statically indeterminate trusses. L. M. H. Navier was the first to use the stiffness method of analysis in the problem of two-degree-of-kinematic indeterminacy in 1826. The well-known Castigliano's theorems were presented in 1879.

The application of the stiffness method and the slope deflection method to plane frames originated with A. Bendixen in 1914, and was also used by W. Wilson and G. A. Maney in 1915. A set of linear equations had to be solved before the moment distribution could be determined. The practical method of structural analysis was introduced later; the moment-distribution method was presented by Hardy Cross (1930).

Tachu (Tanaka) Naito at Waseda University introduced the slope-deflection method in Japan in 1922. He was interested in developing a simple procedure for practical use. Naito (1924) analyzed a series of rectangular frames under horizontal forces to study the lateral stiffness of columns and the height of inflection points. He proposed lateral force distribution ratios (D-value) for interior (1.0) and exterior (0.5) columns, and for flexible frames (1.0) and shear walls (8 to 20) and the height of inflection points in columns to determine the bending moment from the known shear. Another important contribution of Naito was the introduction of reinforced concrete shear walls in the Industrial Bank of Japan Building (an 8-story steel reinforced concrete building completed in 1923) as earthquake resisting elements. The effectiveness of structural walls was demonstrated in the 1923 Kanto Earthquake.

Naito's D-value method of structural calculation for frame buildings was further extended by K. Muto at the Imperial University of Tokyo (Architectural Institute of Japan, 1933). Lateral stiffness of columns was theoretically evaluated taking into account (a) flexural stiffness of the column, (b) stiffness of adjacent girders immediately above and below the column, and (c) support conditions at the column base. Story shear was distributed to each column in accordance with its lateral stiffness. The moment distribution of the column was determined by the column shear and the height of inflection point, which was evaluated taking into account (a) the relative location of story, (b) the stiffness of adjacent girders immediately above and below the column, (c) changes in the stiffness of the adjacent girders, and (d) the difference in inter-story height immediately above and below the column. The sum of column end moments at a joint was distributed to girder ends in proportion to the girder stiffness. Various factors were prepared in table format for practical use.

The use of digital computers for the analysis of statically indeterminate structures began in the mid-1960s. The enhanced memory, the increased speed of computations and input-output processes, and the efficient use of graphics made it possible to use digital computers in routine structural design practices. The finite element method for the analysis of continuum structures was made possible in the early 1960s.

3.4 Seismic design in Urban Building Law of Japan

The first Japanese building code, the Urban Building Law, was promulgated in 1919 to regulate buildings and city planning in six major cities. Structural design was specified in Building Law Enforcement Regulations; i.e., quality of materials, allowable stresses of materials, connections, reinforcement detailing, dead and live loads, and method of calculating stresses were specified, but earthquake and wind forces were not. Allowable (working) stress design, whereby the safety factor for uncertainties was considered in the ratio of the strength to the allowable stress of the material, was in common use at this time in the world.

The 1923 Kanto (Tokyo) earthquake (M 7.9) caused significant damage in the Tokyo and Yokohama areas, killing more than 140,000, damaging more than 250,000 houses, and burning more than 450,000 houses. More than 90 percent of the loss in lives and buildings was caused by fire. The damage to reinforced concrete buildings was relatively low although no seismic design regulations were enforced before the earthquake. The damage was observed in reinforced concrete buildings provided with (a) brick partition walls, (b) little shear walls, or constructed with (c) poor reinforcement detailing, (d) short lap splice length, (e) poor beam-column connections, (f) poor construction, or designed with (g) irregular configuration, and (h) poor foundation.

The Building Law Enforcement Regulations were revised in 1924 to require seismic design using seismic coefficients of 0.10 for the first time in the world. From the incomplete measurement of ground displacement at the University of Tokyo, the maximum ground acceleration was estimated to be 0.3 G. The allowable stress in design was one-third to one-half of the material strength in the design regulations. Therefore, the design seismic coefficient 0.1 was determined by dividing the estimated maximum ground acceleration of 0.3 G by the safety factor of 3 of allowable stresses.

3.5 Seismic design in U.S. Uniform Building Code

The first edition of the Uniform Building Code in 1927, a model code in the United States published by the Pacific Coast Building Officials Conference, adopted the seismic coefficient method for structural design in seismic regions based on the experience from the 1925 Santa Barbara, California, earthquake. The seismic coefficient was varied for soil conditions between 0.075 and 0.10; although buildings on soft soil were known to suffer larger damage, this was the first time for a building code to recognize the amplification of ground motion by soft soil.

After the 1933 Long Beach, California, earthquake, seismic design using a seismic coefficient of 0.02 was made mandatory in California by the Riley Act, and higher seismic safety, using a seismic coefficient of 0.10, was made mandatory for school buildings by the Field Act.

The 1935 Uniform Building Code adopted variations in design seismic forces in three seismic zones, recognizing different levels in seismic risk from one region to another.

3.6 Seismic design in Building Standard Law of Japan

The Building Standard Law, applicable to all buildings throughout Japan, was proclaimed in 1950. Technical issues were outlined in the Building Standard Law Enforcement Order (Cabinet Order). Horizontal earthquake force F_i at floor level i was calculated as

$$F_i = Z\ G\ K\ W_i \tag{2}$$

where Z : seismic zone factor (0.8 to 1.0), G : soil-structure factor (0.6 to 1.0), K : seismic coefficient (0.20 to height of 16 m and below, increased by 0.01 for every 4.0 m above), and W_i : weight of story i including live load for earthquake inertia part. Soil-structure factor G was varied for soil conditions and for construction materials; e.g., for reinforced concrete construction, the coefficient was 0.8 on rock or stiff soil, 0.9 on intermediate soil, and 1.0 on soft soil. The seismic zone factor was based on the seismic hazard map prepared by H. Kawasumi from the Earthquake Research Institute at the University of Tokyo and published in 1946.

At this stage, researchers and engineers discussed earthquake resistant building design without knowing the probable intensity and characteristics of design earthquake motions.

4. Accelerograph and response spectrum

The Earthquake Research Institute was established at the University of Tokyo in 1925, taking over the functions of the Earthquake Disaster Prevention Investigation Council. Many new efforts were made to understand earthquake phenomena and also to develop technology to reduce earthquake disasters. M. Ishimoto developed an accelerograph in 1931; accelerograph records were used to study the dominant period of ground motion at different sites, but not for structure response calculation.

K. Suyehiro, first director of the Earthquake Research Institute, was invited by the American Society of Civil Engineers to give a series of lectures on engineering seismology at U.S. universities in 1931 (Suyehiro 1932). He pointed out the lack of information about earthquake ground acceleration and emphasized the importance of developing accelerographs for engineering purposes.

At the U.S. Seismological Field Survey (later known as the U.S. Coast and Geodetic Survey), established in 1932, F. Wenner and H. E. McComb worked on the development of the first strong motion accelerograph (Montana model) in the same year. An accelerograph at Mt. Vernon station measured the motion during the 1933 Long Beach, California, earthquake, but the amplitude exceeded the capacity of the instrument.

Acceleration records of strong earthquake motions were recorded during the 1935 Helena, Montana, earthquake and the 1938 Ferndale, California, earthquake with peak amplitudes of 0.16 to 0.18 G, respectively. The well-known El Centro records were obtained during the 1940 Imperial Valley earthquake. The El Centro records have been studied extensively and considered as standard acceleration records for a long time. An earthquake acceleration signal is not harmonic, but is quite random in nature, containing high-frequency components. Thus acceleration signals differ greatly from displacement signals in terms of frequency content.

M. A. Biot (1933) from the California Institute of Technology suggested in 1933 that earthquake response amplitude of simple systems to transient impulses should vary with their natural periods, and introduced the concept of a response spectrum. He suggested the use of an electric analyzer. Biot (1941), who later went to Colombia University, developed a mechanical analyzer (torsional pendulum) to calculate the response of linearly elastic systems to an arbitrary exciting function; the 1935 Helena, Montana, earthquake and the 1938 Ferndale, California earthquake records were used to develop the first earthquake response spectra. No damping was used in the calculation. He proposed that the undamped response spectrum peaked at 0.2 s with a maximum amplitude of 1.0 G, and decayed inversely proportional to the period of systems. He pointed out that the response amplitudes could be reduced by the effect of hysteresis of a structure in an inelastic range or damping associated with the radiation of kinetic energy to the foundation (K. Sezawa and K. Kanai, 1938).

Biot's finding that the earthquake force decreased with the fundamental period was first recognized in the City of Los Angeles Building Code in 1943; i.e., the design seismic coefficient C_i at floor i was defined as

$$C_i = \frac{0.60}{N + 4.5} \tag{3}$$

where N: the number of stories above the story under consideration. The maximum number of stories was limited to 13. The requirement also indicated the increase of seismic coefficients with the height from the ground reflecting the deflected shape under dynamic excitation. The 1949 edition of UBC specified similar design seismic forces as follows:

$$F_i = Z \frac{0.15}{N_i + 4.5} W_i \tag{4}$$

where, N : number of stories above, Z: seismic zone factor, and W_i: dead and live loads at level i.

The joint committee of the San Francisco section of the American Society of Civil Engineers and the Structural Engineers Association of Northern California recommended a model code in which the design seismic coefficients were determined inversely proportional to the estimated fundamental period of the structure (Joint Committee, 1951) and the lateral force was distributed linearly from the base to the top. The base shear V was defined by the following equation:

$$V = CW$$
$$C = \frac{0.015}{T} \qquad 0.02 \le C \le 0.06 \tag{5}$$

where C : base shear coefficient, W : sum of dead and live load, and T : natural period of a building evaluated by a simple expression.

The period effect on the amplitude of seismic design forces was not considered in Japan until 1981.

5. System ductility

With the development of digital computers in the late 1950s and with the accumulation of strong motion records, it became possible to calculate linearly elastic as well as nonlinear response of simple structural systems under strong earthquake motions. N. M. Newmark made a significant contribution to earthquake engineering and structural mechanics by developing in 1959 a numerical procedure to solve the equation of motion on digital computers (Newmark, 1959). This method is extensively used in current response analysis programs.

5.1 Newmark's design criteria

Veletsos and Newmark (1960) reported the relation between the maximum response of linearly elastic and elasto-plastic simple systems under earthquake ground motions; i.e., for the linearly elastic and elasto-plastic systems having the same initial period, the strain energy stored at the maximum response was comparable in a short period range and the maximum response displacement amplitudes were comparable in a long period range. On the basis of their observations, Newmark proposed that an elastic-plastic single-degree-of-freedom (SDF) system having ductility capacity μ (ultimate deformation divided by the yield deformation) should be provided with minimum base shear coefficient C_y to resist a ground motion that produced elastic response base shear coefficient C_e ;

$$C_y = \frac{C_e}{\sqrt{2\mu - 1}} \qquad \text{for short period systems} \tag{6}$$

$$C_y = \frac{C_e}{\mu} \qquad \text{for long period systems} \tag{7}$$

The elastic base shear coefficient can be found from the linearly elastic response spectra of an earthquake motion; the plot of maximum response amplitudes with respect to the elastic period of systems for different damping factors. A structure could be designed for smaller resistance if the structure could deform much beyond the yielding point. "Ductility" became an important word in seismic design and a large emphasis was placed on developing structural detailing to enhance deformation capability.

Newmark's design rules opened a new direction in seismic design by providing a means to define the lateral resistance required for survival of a structure. For the precise application of Newmark's rules, plastic hinges in a multi-story building must simultaneous yield to form a plastic mechanism. Due care must be exercised for the concentration of plastic deformation at limited localities where early yielding develops during earthquakes.

Blume, Newmark and Corning (1961) wrote a "classic" design manual for multistory reinforced concrete buildings, published by the Portland Cement Association. The manual was the state of the art in earthquake engineering and earthquake resistance for reinforced concrete buildings. The design was based on the 1959 SEAOC recommendations in terms of design earthquake forces, but the design of reinforced concrete was based on the allowable stress procedure of the 1956 American Concrete Institute Building Code; the ultimate strength design procedure was treated as alternative method in the code. It should be noted that the manual discussed the advantage of weak-beam strong-column systems. Evaluation of strength, ductility and energy absorption of reinforced concrete members was discussed, elaborating on such issues as the moment-curvature relation of sections to failure, the effect of compressive longitudinal reinforcement and confining reinforcement on deformation capacity, the interaction of ultimate moment and axial force, and the effect of reversed loading. Good reinforcement details were suggested to improve ductility and energy absorption.

Studies on earthquake response of structural systems slowed down in Japan during and after the Second World War. As economic conditions stabilized and improved during and after the Korean War (1950-1953), some research funds were made available to research communities. The Strong Motion Accelerograph Committee (SMAC) was formed in 1951 and developed a series of SMAC-type seismometers that were installed throughout the country. The Strong Earthquake Response Analysis Computer (SERAC) was built at the University of Tokyo (Strong Earthquake Response Analysis Committee, 1962) under the leadership of K. Muto. This was an electric analog computer capable of calculating the elasto-plastic response of up to a five-mass spring system. This analog computer was replaced as the result of the development of digital computers approximately five years later, but produced useful information about the nonlinear earthquake response of multi-degree-of-freedom systems, data that would be of benefit for the construction of high-rise buildings in an earthquake prone country such as Japan. The reduction of design seismic forces relying on ductility was not considered in Japan until 1981.

5.2 Nonlinear effect in SEAOC Code

The Seismological Committee of the Structural Engineers Association of California (SEAOC) published a seismic design model code in 1957, which was formally adopted in 1959 (Seismological Committee, 1959). The code represented the state of the art in earthquake engineering at the time. The minimum design base shear V for buildings was expressed as

$$V = KCW \tag{8}$$

where horizontal force factor K: the type of structural systems, and W: the weight of a building. Seismic coefficient C is inversely proportional to the cubic root of fundamental period T of structures, but is limited to 0.10;

$$C = \frac{0.05}{\sqrt[3]{T}} \tag{9}$$

The code recognized different performance of structural systems during an earthquake. Horizontal force factor K was 1.33 for buildings with a box system, and 0.80 for buildings with a complete horizontal bracing system capable of resisting all lateral forces. The latter system included a moment resisting space frame which, when assumed to act independently, was capable of resisting a minimum of 25% of the total required lateral force. K was 0.67 for buildings with a moment resisting space frame which when assumed to act independently of any other more rigid elements was capable of resisting 100% of the total required lateral forces in the frame alone, and 1.0 for all other building frame systems.

The commentary of the 1967 SEAOC code explicitly stated that "... structures designed in conformance with the provisions and principles set forth therein should be able to:

1. Resist minor earthquakes without damage;
2. Resist moderate earthquakes without structural damage, but with some nonstructural damage;
3. Resist major earthquakes, of the intensity of severity of the strongest experienced in California, without collapse, but with some structural as well as nonstructural damage."

This concept has been generally accepted by researchers and engineers in the world.

Figure 2 shows schematically the expected performance of a building under earthquake motions. The level of minimum lateral resistance should be determined (a) to control the serviceability of buildings from minor but more frequent earthquake motions and (b) to protect the occupants' life by limiting nonlinear deformation from very rare but maximum probable earthquake motions. Architectural elements, such as non-structural curtain walls, partitions and mechanical facilities, must be protected for the continued use of a building after more frequent earthquakes. It should be noted that the structure of higher resistance suffers no damage from infrequent earthquakes while the structure of low resistance suffers some structural damage and associated nonstructural damage, which must be repaired before use is resumed.

The 1966 SEAOC code implicitly assigns expected ductility of a building according to its framing system, and much larger variation was adopted in horizontal force factor K. More strict structural detailing requirements were specified for framing systems using a small horizontal force factor.

5.3 Allowable stress design to ultimate strength

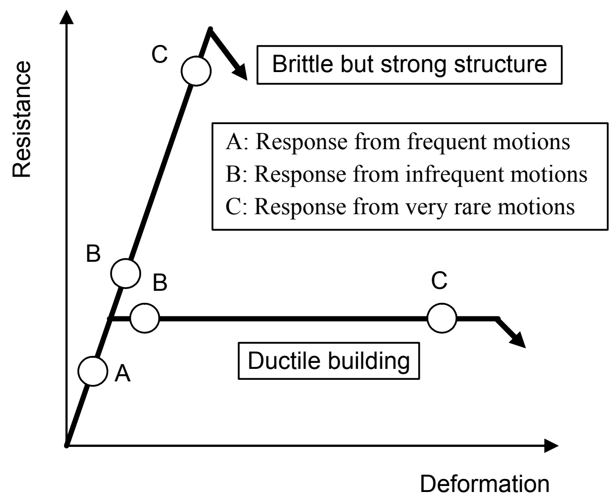

Fig. 2 Performance objectives of building.

design

The limitation of allowable stress design procedure based on single material safety factor was gradually noted; for example, (a) the attainment of material strength at a locality did not lead to the failure of the structural member, (b) the safety margin at failure after stresses in section reached the allowable stress varied with the amount of reinforcement, (c) even after the attainment of member strength, some members could continue to support the applied load with plastic deformation, (d) acceptable damage levels might vary with the importance of members and with the different uncertainties of loading conditions; e.g., dead and live loads.

The Japan Architectural Standard, which was issued in 1947, proposed two levels of allowable stresses for the structural calculation; i.e., one for permanent loading and the other for extraordinary loading. Much larger allowable stresses were specified for extraordinary loading with a corresponding increase in the amplitude of design forces. Similar efforts were made in Europe during the Second World War.

The ultimate strength of reinforced concrete members was extensively studied in the 1950s and 1960s. Flexural strength of reinforced concrete members with and without axial loads could be estimated with reasonable confidence. Some brittle modes of failure were identified and such modes were to be avoided in design either by using higher design forces or by using low capacity reduction factor. Statistical variation of material strengths in practice and amplitudes of loads, reliability of strength evaluations, consequence of member failure were considered in the load-factor and capacity-reduction factor format. The American Concrete Institute (1956 and 1963) adopted the ultimate strength design procedure as an alternative procedure to the on-going allowable stress design in 1956, and then shifted from the allowable stress design to the ultimate strength design in 1963. The Euro-International Concrete Committee (1964), founded in 1953, treated the design problems in a more rigorous probabilistic manner and recommended limit states design on the basis of the ultimate strength of members.

6. Nonlinear response analysis of buildings

With knowledge to estimate ultimate strength of reinforced concrete members, the behavior under load reversals was investigated. The response of reinforced concrete sections under alternating moment was calculated by Aoyama (1964); the effect of longitudinal reinforcement on hysteretic behavior was demonstrated. The response analysis of reinforced concrete under load reversals is difficult because the force-deformation relation varies with the loading history and because the damage spreads along the member.

6.1 Nonlinear earthquake response analysis of buildings

With an accumulation of experimental data in the laboratory, more realistic resistance-deformation relations, commonly known as hysteresis models, were formulated for structural members; e.g., Clough model (Clough and Johnston 1966) and Takeda model (Takeda et al. 1970). Mathematical models to represent the damage distribution of a member were studied. Methods to calculate the nonlinear earthquake response of structures were developed by Clough et al. (1965) and Giberson (1967). Giberson's one-component model, in which all inelastic deformation was assumed to concentrate at the member ends, is commonly used in earthquake response analysis. General-purpose computer software was developed by many researchers; e.g. DRAIN 2D program by Powell in 1973.

The first U.S. shake table was installed in 1967 at the University of Illinois at Urbana-Champaign, and later at the University of California at Berkeley. Takeda et al. (1970) tested reinforced concrete columns on the Illinois earthquake simulator and demonstrated that the nonlinear response of reinforced concrete columns under earthquake excitation could be reliably simulated if a realistic force-deformation relation was used in the analysis. Otani and Sozen (1973) tested three-story one-bay reinforced concrete frames and showed that the response of such frames could be simulated with the use of reliable member hysteresis and damage distribution models.

It is technically difficult to test structural members under dynamic conditions in a laboratory. The speed of loading is known to influence the stiffness and strength of various materials. Mahin and Bertero (1972) reported the findings of dynamic test of reinforced concrete members as follows: (a) high strain rates increased the initial yield resistance, but caused small differences in either stiffness or resistance in subsequent cycles at the same displacement amplitudes; (b) strain rate effect on resistance diminished with increased deformation in a strain-hardening range; and (c) no substantial changes were observed in ductility and overall energy absorption capacity. Therefore, the strain rate effect was judged to be small in the case of earthquake response.

A full-scale seven-story reinforced concrete building with a structural wall was tested using the computer-online pseudo-dynamic testing method at the Building Research Institute of the Ministry of Construction in 1980, as part of U.S.-Japan cooperative research utilizing large testing facilities. Member and sub-assemblage specimens were tested before the full-scale specimen. When all the information concerning members and full-scale test results was carefully examined in formulating a mathematical model, the calculated overall structural as well as member response was shown to agree well with the response observed using the state of the art at the time (Otani, et al. 1985).

6.2 Seismic design in ATC-03

When the review work of the 1959 SEOC was initiated in 1970, the 1971 San Fernando, California, Earthquake hit suburban areas of Los Angeles, causing significant damage to hospital buildings. It was recognized that the potential of major earthquake damage increases with the increase in population and urban densities. The Applied Technology Council (ATC) initiated a project to develop tentative but comprehensive seismic design provisions in 1974 under research contracts with the National Science Foundation and National Bureau of Standards in the U.S. The first comprehensive seismic design document was drafted in 1976 based on modern earthquake engineering principles (Applied Technology Council, 1978). Many new concepts were introduced; e.g., (a) more realistic ground motion intensities, (b) effect of distant earthquakes on long-period buildings, (c) response reduction factors according to toughness and damping in inelastic range, (d) introduction of seismic hazard exposure groups, and (e) seismic performance categories.

Ground intensities and seismic indices were defined by the peak ground acceleration and the effective peak velocity-related acceleration at the construction site. Three seismic hazard exposure groups were defined. Group III buildings having essential facilities that are necessary for post-earthquake recovery, should have the capacity to function during and immediately after an earthquake. Group II buildings have a large number of occupants or occupants of restricted mobility. Group I buildings are all other buildings not belonging to Group III or Group II. Allowable story drift was specified for the seismic hazard exposure group to control the damage level (allowable drift ratio is 0.01 for group III, and 0.015 for Groups II and I).

A seismic performance category was assigned to each building. The analysis procedure, design and detailing requirements were specified for the seismic performance category. Equivalent lateral force procedure and modal analysis procedure were outlined in the document. The design base shear V of a building in the equivalent lateral force procedure was defined as

$$V = \frac{1.2\,A_v\,S}{R\,T^{2/3}}W \tag{10}$$

where W : total gravity load of the building, A_V : effective peak velocity-related acceleration, S : soil profile coefficient (1.0 for hard soil, 1.2 for intermediate soil, and 1.5 for soft soil), R : response modification factor (4.5 for reinforced concrete bearing wall system, 5.5 for building frame system with reinforced concrete shear walls, 7.0 for reinforced concrete moment resisting frame system, and 8.0 for dual system with reinforced concrete shear walls), and T : fundamental period of the building. The deflection of a building was first calculated as elastic deformation under the design seismic forces, and was then multiplied by the amplification factor, which was slightly smaller than the response modification factor. Soil-structure interaction must be considered in determining design earthquake forces and corresponding displacement of the building.

The concept of ATC03 was further extended and adopted in the Uniform Building Code.

7. New seismic design concepts

Ultimate strength design refers to the ultimate strength of structural members, but does not represent the ultimate strength of a structural system. The attainment of ultimate strength in a few members will not lead to the collapse of the structure. Design concept, based on more explicit formation of a collapse mechanism as the strength of a structural system, emerged in the mid-1970s. Although inelastic response of structural members is assumed in design, the inelastic deformation of the members is not realistically estimated in structural analysis. Recent design procedures in the world consider inelastic response of structural members explicitly.

7.1 Capacity design

An integrated design procedure called Capacity Design was developed for reinforced concrete buildings in New Zealand under the leadership of T. Paulay (Paulay, 1970). The capacity design philosophy is a general design concept to realize the formation of an intended yield mechanism.

(1) Required resistance

The required level of horizontal force resistance should be determined taking into consideration, (a) characteristics of the maximum intensity ground motion expected at the construction site, and (b) acceptable deformation at expected yield hinge regions of a structure.

(2) Desired yield mechanism

The weak-beam strong-column mechanism has been preferred by many structural engineers; i.e., a moment-resisting frame develops yield hinges at the end of girders and at the base of first-story columns and structural walls to form collapse mechanism (**Fig. 3**). The earthquake input energy can be quickly dissipated by fat and stable hysteresis of flexural yielding beams. For a given displacement of a structure, the ductility demand at yield hinges in the weak-beam strong-column structure is minimum because plastic deformations are uniformly distributed throughout the structure. It is also true that the deformation capacity is reasonably large in girder members where no axial force acts; on the other hand, the formation of a plastic hinge at the base of the first story column is not desirable because large deformation capacity is hard to develop at the locality due to the existence of high axial load. Some extra moment resistance should be provided at the base of the first story columns to delay the yield hinge formation. Local story mechanism as shown in **Fig. 3** should be avoided,

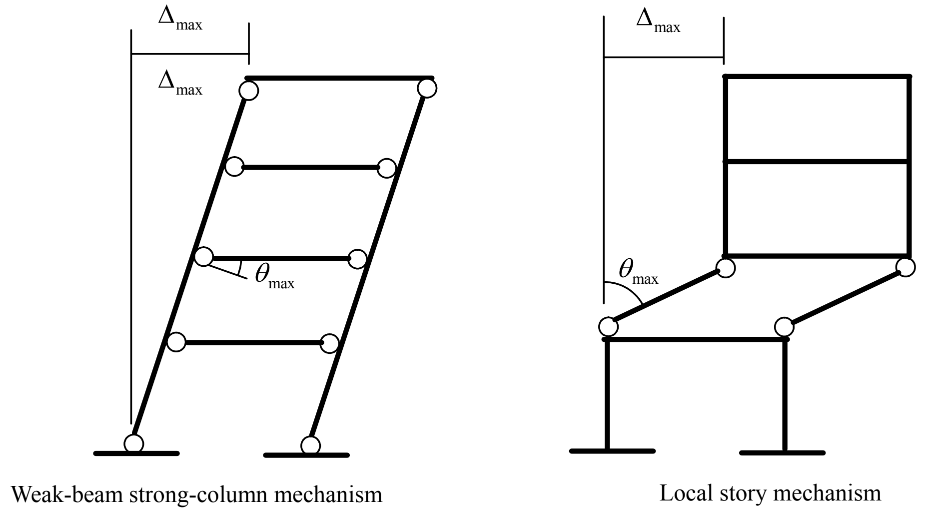

Fig. 3 Weak-beam strong-column mechanism.

but minor yielding of some columns in a story should be tolerated as long as the column can support the gravity load.

(3) Resistance at yield hinges

A nonlinear analysis (commonly known as push-over analysis) under monotonically increasing lateral forces is carried out until the planned yield mechanism (normally the weak-beam strong-column yield mechanism) develops the acceptable damage at critical regions. The lateral force distribution is taken similar to the first mode shape. The contribution of higher modes should be considered, especially in the displacement response of high-rise buildings, in selecting the distribution pattern of lateral forces for high-rise buildings. The resistance at the yield mechanism formation should be greater than the required resistance.

(4) Assurance of planned yield mechanism

In order to ensure the planned yield mechanism during an earthquake, extra resistance should be provided in the region where yielding is not desired and against undesired brittle modes of failure, such as shear failure and bond splitting failure along the longitudinal reinforcement. The members and regions that are not part of the planned yield mechanism should be protected from the action calculated in the pushover analysis by the following reasons:

(a) Horizontal force distribution during an earthquake can be significantly different from that assumed in the pushover analysis due to higher mode contribution;

(b) Actual material strength at the expected yield hinge can be higher than the nominal material strength used in design; therefore, the actions in non-yielding members may be increased at the formation of a yield mechanism with enhanced resistance at each yield hinge;

(c) Additional contribution of slab reinforcement to the bending resistance of a girder with deformation; i.e., the width of slabs effective to the flexural resistance of a yielding girder becomes wider with a widening of flexural cracks at the critical section;

(d) Bi-directional earthquake motion develops higher actions in vertical members than uni-directional earthquake motion normally assumed in a structural design; and

(e) Actual amount of reinforcement may be increased from the required amount for construction reasons.

The level of additional resistance must be determined in the development of design requirements using a series of nonlinear response analyses of typical buildings under credible earthquake motions.

(5) Limitation

When the survival of a structure under a severe earthquake motion is the design objective, the weak-beam strong-column design is probably most desirable. However, it should be noted that the weak-beam strong-column mechanism requires a significant number of localities to be repaired after an earthquake. This is the problem after an infrequent medium-intensity earthquake; i.e., the repair of yielding and associated damage at many localities results in significant cost for continuing use.

7.2 1981 Building Standard Law Enforcement order

The Ministry of Construction of Japan organized an integrated technical development project entitled "Development of New Earthquake Resistant Design (1972-1977)." The Enforcement order of the Building Standard Law was revised in July 1980 following the recommendations of the development project and was enforced from June 1981. The major revision points are listed below.

(1) Design and construction of a building is made

possible up to 60 m in height; the design and construction of buildings taller than 60 m must be approved by the Minister of Construction,

(2) Additional requirements were introduced in structural calculation; (a) story drift, rigidity factor and eccentricity factor under design earthquake forces, (b) examination of story shear resisting capacity at the formation of a collapse mechanism under lateral forces, (c) alternative simple procedures for buildings with abundant lateral shear resisting capacity,

(3) Design earthquake forces were specified (a) by story shear rather than horizontal forces, (b) as a function of the fundamental period of the structure, (c) at two levels (allowable stress design and examination of story shear resisting capacity), and (d) also for the underground structures, and

(4) Strength of materials was introduced for the calculation of ultimate member resistance in estimating story shear resisting capacity.

(1) Design elastic story shear

The seismic (elastic response) story shear coefficient C_i is calculated by :

$$C_i = ZR_tA_iC_o \tag{11}$$

where, Z : seismic zone factor (0.7 to 1.0 in Japan), R_t : vibration characteristic factor, A_i : factor representing vertical distribution of the seismic story shear coefficient, C_0 : basic base shear coefficient (0.2 for conventional allowable stress design and 1.0 for the examination of story shear resisting capacity). The vibration characteristic factor R_t represents the shape of design acceleration response spectrum:

$$\begin{aligned} &R_t=1.0 \quad for T < T_c \\ &R_t=1.0-0.2\{\frac{T}{T_c}-1\}^2 \quad for T_c \le T < 2T_c \\ &R_t=1.6\frac{T_c}{T} \quad for 2T_c \le T \end{aligned} \tag{12}$$

where, T_c : dominant period of subsoil (0.4 s for stiff sand or gravel soil, 0.6 s for other soil, and 0.8 s for alluvium mainly consisting of organic or other soft soil); T : natural period of the building. The natural period of a reinforced concrete building may be estimated by the following simple expression:

$$T = 0.02H \tag{13}$$

where, H : total height in m. The coefficient A_i defines the distribution of design story shear along the height of a building:

$$A_i = 1 + (\frac{1}{\sqrt{\alpha_i}} - \alpha_i)\frac{2T}{1+3T} \tag{14}$$

where $\alpha_i = \sum W_i / \sum W_1$, $\sum W_i$: total of dead and live loads above story i, and $\sum W_1$: total dead and live loads of the building.

(2) Serviceability requirements

Conventional earthquake forces are the elastic design story shear using standard base shear coefficient C_0 of 0.20. The stress in structural members under gravity loads and the conventional earthquake forces should not exceed the allowable stress of materials. The story drift angle under the conventional earthquake forces must be not more than 1/200 of the story height and the story drift limit can be increased to 1/120 if the damage of the structure and nonstructural elements can be controlled.

(3) Strength requirements

Each story of a building must retain a story shear resisting capacity greater than the required story shear resisting capacity Q_{un} defined below:

$$Q_{un} = D_sF_{es}C_i\sum W_i \tag{15}$$

where, D_s : structural characteristic factor, representing the ductility of hinging members of the story, F_{es} : structural configuration factor, representing the distribution of stiffness and mass in a story, C_i : story shear coefficient, and $\sum W_i$: total dead and live loads above story i.

Structural characteristic factor D_s , a reduction factor of the required strength from elastic design shear, may be defined for each story, considering the deformation rank of hinging members at the formation of a yield mechanism. The deformation rank is defined by (a) ratio of shear stress to concrete strength, (b) tensile reinforcement ratio, (c) ratio of axial stress to concrete strength, and (d) shear span to depth ratio. Structural characteristic factors of reinforced concrete buildings vary from 0.30 for ductile structures to 0.55 for non-ductile structures.

Structural configuration factor F_{es} considers the distribution of stiffness along the height of the structure and also the eccentricity of mass center with respect to the center of rigidity in a floor. The structural configuration factor is calculated as the product of factors F_s and F_e representing the irregularity of stiffness distribution in height and eccentricity in plan, respectively, as given below:

$$F_{es} = F_e\,F_s \tag{16}$$

7.3 Capacity spectrum method

The new structural design procedure was introduced in the existing Building Standard Law Enforcement Order in 2000 for the evaluation and verification of performance (response) at a given set of limit states under (a) gravity loads, (b) snow loads, (c) wind and (d) earthquake forces. In addition, structural specifications were prescribed for the method of structural calculation, the quality control of construction and materials, the durability of buildings, and the performance of nonstructural elements.

(1) Design limit states

The performance of a building is examined at the two limit states under two levels of design earthquake motions; i.e., (a) damage-initiation limit state and (b) life-safety limit state.

The properties should be protected under normal gravity loading and in events that may occur more than once in the lifetime of the building; i.e., the damage must be prevented in structural frames, members, interior and exterior finishing materials in events with return periods of 30 to 50 years. The damage-initiation limit state is attained when the allowable stress of materials is reached in any member or when the story drift reaches 0.5 percent of the story height at any story. The initial elastic period is used for a structure. The allowable stresses of concrete and reinforcement are two-thirds nominal compressive strength and yield stress, respectively.

For the protection of human life, no story of the building should collapse even under extraordinary loading conditions, such as an event with a return period of several hundred years. The life-safety limit state is attained when the structure cannot sustain the design gravity loads in a story under additional horizontal deformation; i.e., when a structural member has reached its ultimate deformation capacity. The ultimate deformation of a member must be calculated as the sum of flexure and shear deformations of the member and deformation resulting from the deformation in the connection to adjacent members.

(2) Design earthquake forces

The seismic design response acceleration spectrum $S_A(T)$ of free surface ground motion at a 5% damping factor is represented as follows;

$$S_A(T) = Z \cdot G_s(T) \cdot S_0(T) \tag{17}$$

where Z : seismic zone factor, $G_s(T)$: amplification factor by surface geology, $S_0(T)$: response spectral acceleration ordinate of ground motion at exposed engineering bedrock, and T : period of a building expressed in seconds at the damaged state. Seismic zone factor Z evaluates the relative difference in expected intensities of ground motion. Two levels of ground motion are defined; i.e., (a) Large earthquake: largest motion in 500 years, and (b) Intermediate earthquake: 10th largest motion in 500 years. The acceleration response spectrum is specified at exposed engineering bedrock. The design spectrum $S_o(T)$ at exposed engineering bedrock is given by **Fig. 4** for the life-safety limit state: The design spectrum for the damage-initiation limit state is to be reduced to one-fifth of the spectrum for the life-safety limit state.

The ground motion of an earthquake is significantly affected by the surface geology. The nonlinear amplification of ground motion by surface geology is evaluated using the geological data at the site and an equivalent linear multi-mass shear-spring model. The shear modulus reduction factors and damping factors are specified for cohesive and sandy soils at various shear strain levels.

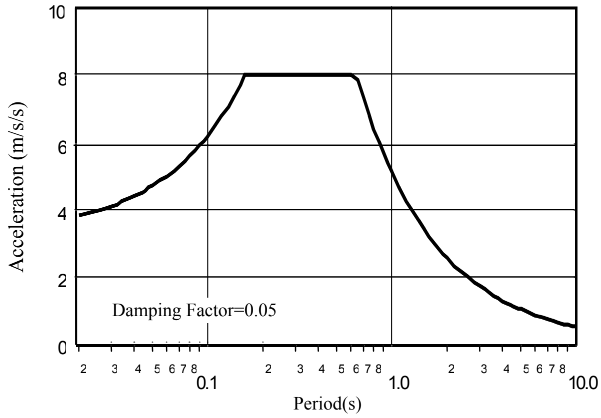

Fig. 4 Design earthquake acceleration response spectrum on exposed engineering bedrock for life-safety limit state.

(3) Demand spectrum

The design spectrum is transformed to "Demand Spectrum" by plotting a diagram with design spectral acceleration $S_A(T,h)$ in the vertical axis and spectral displacement $S_D(T,h)$ in the horizontal axis (**Fig. 5**). When a viscous damping of a linear system is small, the response spectral displacement is approximated by the expression below:

$$S_D(T,h) = \left(\frac{T}{2\pi}\right)^2 S_A(T,h) \tag{18}$$

Demand spectrum for an equivalent damping ratio h_{eq} can be obtained by reducing the spectral acceleration and displacement ordinates at 0.05 damping factor by the following factor F_h ;

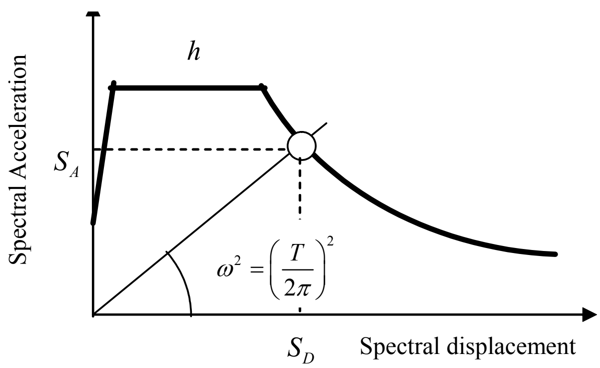

Fig. 5 Formulation of demand spectrum of design earthquake motion.

$$F_h = \frac{1.5}{1+10h_{eq}} \tag{19}$$

The damping factor varies according to the amount of damage in constituent members in a structure.

(4) Capacity spectrum

A multi-story building structure is reduced to an equivalent single-degree-of-freedom (SDF) system using the results of a nonlinear static analysis under constant-amplitude gravity loads and monotonically increasing horizontal forces (often called a "pushover analysis"). The deflected shape of the pushover analysis is assumed to represent the first-mode shape of oscillation.

If a structure responds in the first mode to the ground motion having spectral acceleration $S_A(T_1, h_1)$ and displacement $S_D(T_1, h_1)$ at the first-mode period T_1 and damping factor h_1. For the mode shape vector normalized to the roof-level displacement, the maximum roof displacement $D_{R1\max}$ and the maximum first-mode base shear $V_{B1\max}$ are calculated as follows:

$$D_{R1\max} = \Gamma_1 \, S_D(T_1, h_1) \tag{20}$$

$$V_{B1\max} = M_1 \, S_A(T_1, h_1) \tag{21}$$

where M_1: effective modal mass, and Γ_1: first mode participation factor.

$$M_1 = \Gamma_1 \{1\}^T [m] \{\phi\}_1 \tag{22}$$

$$\Gamma_1 = \frac{\{\phi\}_1^{\ T} [m] \{1\}}{\{\phi\}_1^{\ T} [m] \{\phi\}_1} \tag{23}$$

where $\{\phi\}_1$: first-mode shape vector, $[m]$: lumped floor mass matrix (diagonal matrix), and $\{1\}$: vector with unity elements.

The spectral acceleration $S_A(T_1, h_1)$ and displacement $S_D(T_1, h_1)$ required to develop maximum base shear $V_{B\max}$ and roof displacement $D_{R\max}$ of a structure can be defined as follows:

$$S_A(T_1, h_1) = \frac{V_{B\max}}{M_1} \tag{24}$$

$$S_D(T_1, h_1) = \frac{D_{R\max}}{\Gamma_1} \tag{25}$$

A structure is assumed to respond elastically to the ground motion using the secant stiffness and equivalent damping factor defined at the maximum base shear and roof displacement. For each point on the base shear-roof displacement relation of a structure under monotonically increasing horizontal forces, the corresponding acceleration and displacement spectral ordinates $S_A(T_1, h_1)$ and $S_D(T_1, h_1)$ can be plotted as shown in **Fig. 6**. The relation is called the "capacity spectrum" of the structure.

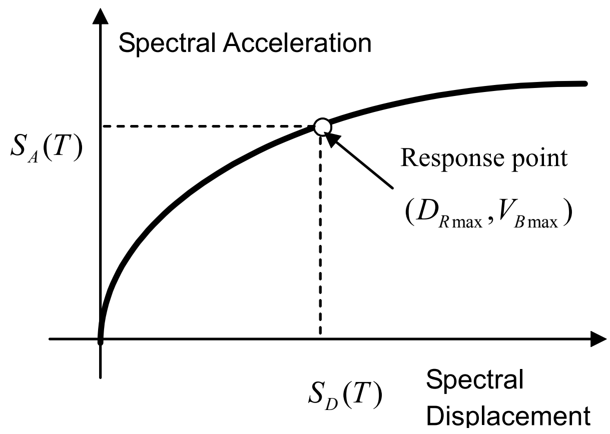

Fig. 6. Capacity spectrum using spectral acceleration $S_A(T)$ and displacement $S_D(T)$

(5) Equivalent damping ratio

An equivalent viscous damping ratio h_{eq} at a damage state is defined by equating the energy dissipated by hysteresis of a nonlinear system and the energy dissipated by a viscous damper of a linearly elastic system under resonant steady-state vibration:

$$h_{eq} = \frac{1}{4\pi} \frac{\Delta W}{W} \tag{26}$$

where ΔW : hysteresis energy dissipated by a nonlinear system during one cycle of oscillation, and W : elastic strain energy stored by a linearly elastic system at the maximum deformation (**Fig. 7**). For the damage-initiation limit state, a constant damping ratio of 0.05 is prescribed because the state of a structure remains linearly elastic at this stage. The equivalent damping ratio must be effectively reduced to correlate the maximum response of an equivalent linear system and a nonlinear system under random earthquake excitation.

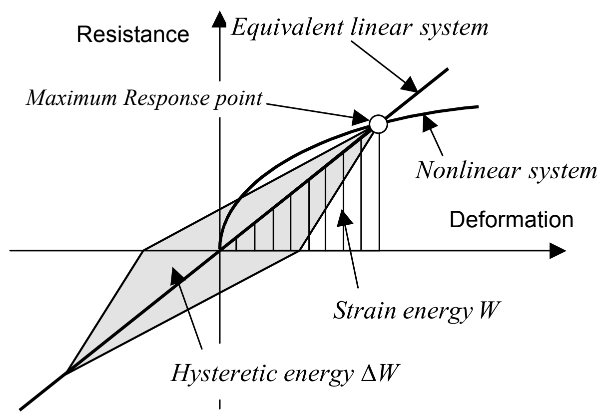

Fig. 7 Equivalent viscous damping ratio for hysteresis energy dissipation.

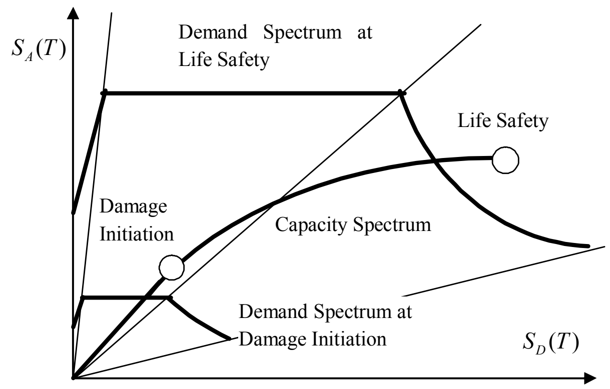

Fig. 8 Demand spectra and capacity spectrum at damage initiation and life safety limit states.

(6) Performance judgment

The performance of a structure under a given design earthquake motion is examined by comparing the capacity spectrum of the structure and the demand spectra of design earthquake motions evaluated for equivalent damping factors at the two limit states. Spectral acceleration of a structure at a limit state should be higher than the corresponding acceleration of the demand spectrum using the equivalent damping ratio.

8. Lessons learned from earthquakes

Earthquake engineering is not a pure science, but has been developed through the observation of failure of structures during earthquakes. The sole aim of earthquake engineering has been not to repeat the same mistakes in the event of future earthquakes.

This section reviews the observation of damage of man-made construction, with emphasis on the damage to reinforced concrete buildings. Those defects found in existing constructions should be identified by vulnerability assessment and retrofitted for safety in the event of future earthquakes.

8.1 Structural damage associated with system faults

Similar failure patterns of buildings have been repeatedly observed in the investigation of past earthquake damage. Design requirements have been modified or added for the protection of new construction. However, older structures, designed and constructed using outdated technology, are susceptible to the same patterns of damage during future earthquakes.

(1) Heavy structures

Inertia forces in horizontal and vertical directions are developed with vibration of a structure. Vertical inertia forces are developed by the vertical vibration of a structure caused by the vertical ground motion and also by the vibration of floor slabs. The dominant part of structural damage is caused by horizontal inertia forces associated with lateral vibration of the structure. The amplitude of inertia forces is proportional to the mass of a structural part in vibration and the response acceleration developed at the point. Heavy structures, such as adobe houses and reinforced concrete construction, attract larger inertial forces during an earthquake. Minimum resistance should be provided to resist horizontal and vertical inertia forces corresponding to the weight of a structure.

(2) Period of vibration

Acceleration is an important index in engineering. Although the acceleration signal of an earthquake ground motion appears to be random, the signal contains special dominant periods of vibration, representing the characteristics of surface geology at the construction site. The acceleration amplitude of ground motion is generally large in a period range less than 0.5 to 1.0 s, and it decays with the length of periods. Therefore, the acceleration response, corresponding to the inertia forces, is generally large for short period structures. For a given duration of an earthquake motion, the short period structure is subjected to more cycles of oscillation; i.e., the short period structure is generally more susceptible to damage unless larger resistance is provided.

(3) Strength and deformation capacity

A structure does not always fail immediately when the action reaches the strength (maximum resisting capacity) of a structure. A structure collapses when the deformation capacity is reached in vertical load carrying members, such as columns and walls. The location of damage can be controlled by selecting weak regions of a structure in design planning. A large deformation capacity after reaching the strength, commonly known as ductility, can be built into weak structural members so that the collapse can be delayed even after significant structural damage is developed.

The brittle modes of failure should be prevented in vertical load carrying members. If the brittle modes of failure cannot be corrected in construction, then higher strength must be provided and also the mass of the construction should be reduced.

The structural damage of a building with high lateral resistance (stiffness and strength) is likely to be smaller under frequent minor earthquakes than that of a building with low resistance, regardless of the deformation capacity. Therefore, a certain minimum resistance is necessary for the continued operation of buildings after frequent earthquakes.

(4) Progressive collapse

When a vertical member, such as a column or a structural wall, fails in a brittle mode, the shear carried by the member must be resisted by the other vertical members of the same story. The additional shear often triggers brittle failure of the other members because the structural members are normally designed under the same specification; i.e., if a member fails in a brittle manner, the other members may fail in a similar mode. Collapse of a building in a story occurs by progressive brittle failure of vertical members.

Failure of vertical members does not simply results in the reduction of lateral resistance, but also results in loss of vertical load carrying capacity. The gravity load supported by the failing member must be transferred to adjacent vertical members. The failure of gravity load transfer causes partial collapse around the failing vertical member.

(5) Concentration of damage

The concentration of structural deformation and associated damage to limited localities should be avoided if the deformation capacity at expected damage locations is limited, especially in reinforced concrete buildings. Collapse of a building is normally caused by the failure of vertical load carrying members of a story. In order to protect vertical members in a multi-story construction, they should be provided with higher strength than connecting horizontal members so that damage should be directed to the horizontal members.

(6) Vertical irregularities

When the stiffness and associated strength are abruptly reduced in a story along the height, earthquake-induced deformations tend to concentrate at the flexible and/or weak story. The concentration of damage in a story leads to large deformation in vertical members. The excessive deformation in vertical members often leads to the failure of these members and the collapse of the story.

Soft/weak first stories are especially common in multi-story residential buildings in urban areas, where the first story often is used for open space, commercial facilities or garages. For example, structural walls that separate residential units in levels above may be discontinued in the first story to meet flexible usage requirements. The first-story columns during strong earthquake shaking must resist a large base shear, inevitably leading to large story drift concentrated in that story.

(7) Horizontal irregularities

If, for example, structural walls are placed on one side of a building while the other side has open frames, the eccentricity between the centers of mass and resistance causes torsional vibration during an earthquake. Larger damage develops in members away from the center of resistance. The structural wall is effective reducing lateral deformation and resisting large horizontal forces, especially when they are distributed in plan.

(8) Contribution of nonstructural elements

Nonstructural elements, such as masonry or concrete infill walls and stairways, are normally disregarded in structural analysis although they can contribute significantly to the stiffness of the framing system. The existence of these high-stiffness nonstructural elements can cause irregular stiffness distributions in plan or along the height.

Nonstructural elements are commonly neglected in modeling and analysis in design calculations, but are placed for the purpose of building function, for example, partition walls. When stiff and strong nonstructural elements are placed in contact with structural elements, the interaction can leads to the damage in nonstructural and structural elements. A typical example is a captive column, where deformable length is shortened by spandrels directly attached to the column.

(9) Pounding of adjacent buildings

Pounding of adjacent buildings causes structural damage. Proper distance should be maintained between adjacent buildings. In the case of a series of buildings constructed side-by-side in some localities, the edge buildings are often pushed outward and suffer severe damage while inner buildings are protected from excessive lateral deformation.

(10) Deterioration with age

Deterioration of structural materials due to aging and aggressive environmental conditions reduces the seismic performance potential of a building. Prior earthquake damage, unless properly repaired and strengthened, has the same effect. It is important either to maintain the structure at regular intervals or follow rigid construction specifications for durability of the structure.

(11) Foundation

The failure of foundations is caused by: (a) liquefaction and loss of bearing or tension capacity, (b) landslides, (c) fault rupture, (d) compaction of soils, and (e) differential settlement. It is normally difficult to design and construct a safe foundation to resist ground movement immediately above the fault rupture. Although foundation failures normally do not pose a life threat, the cost of damage investigation and repair work is extremely high. Therefore, it is advisable to reduce the possibility of foundation failure.

(12) Nonstructural elements

Damage of nonstructural or architectural elements, such as partitions, windows, doors and mechanical facilities, interrupts the use of a building. The cost of repair work on a building is often governed by the replacement of the damaged nonstructural elements, rather than the repair work on structural elements. Damage of nonstructural elements may create a falling hazard for people in, or escaping from, the building; furthermore, fallen elements may block evacuation routes in a severely damaged building.

8.2 Damage in structural members

Failure types of members may be different for columns, beams, walls and beam-column joints. It is important to consider the consequence of member failure on structural performance; e.g., the failure of vertical members often leads to the collapse of a building. Failure modes in flexure and shear of a member and bond failure along the longitudinal reinforcement are reviewed.

(1) Flexural compression failure of columns

A reinforced concrete member subjected to axial force and bending moment normally fails in compression of concrete after the yielding of longitudinal reinforcement; this failure mode is normally called flexural compression failure. The deformation capacity of a column is influenced by the level of axial force in the column and the amount of lateral reinforcement provided in the region of plastic deformation. The level of axial force is limited in design to a relatively low level under the gravity condition. During an earthquake, however, exterior columns, especially corner ones, are subjected to varying axial force due to the overturning moment of a structure; the axial force level in these columns may become extremely high in compression, leading to flexural compression failure. It is often difficult to distinguish shear compression failure and flexural compression failure, as both failures takes place near the column ends and involves concrete crushing. The lateral confining reinforcement can delay the crushing failure of concrete under high compressive stresses.

(2) Shear failure of columns

The most brittle mode of member failure is shear. Shear force causes tensile stress in concrete in the diagonal direction to the member axis. After the concrete cracks under the tensile stress, the stress must be transferred to the lateral reinforcement. Brittle shear failure occurs in the diagonal tension mode when the minimum amount of lateral reinforcement (size, spacing and strength of shear reinforcement) is not provided in the member.

When the minimum amount of lateral reinforcement is provided in a member, the shear failure is developed in the form of diagonal compression failure of concrete after the yielding of lateral reinforcement. This mode of failure is not as brittle as the diagonal tension failure, but the deformation capacity is limited. If an excessive amount of lateral reinforcement is provided, diagonal compression failure of concrete takes place prior to the yielding of lateral reinforcement. Therefore, there is an upper limit in the amount of lateral reinforcement effective for shear resistance. After the compressive failure of concrete, the vertical load carrying capacity of the column is lost, leading to the collapse in the story.

Because the lateral reinforcement resists tensile force under shear, the ends of rectilinear lateral reinforcement should be anchored in the core concrete with 135-degree bend, or they should be welded together. When a reinforcing bar is bent, permanent plastic deformation takes place at the bend and the region becomes less ductile. The reinforcing steel capable of developing high toughness and ductility before fracture must be used for lateral reinforcement.

(3) Shear failure of flat-plate construction

A flat plate floor without column capitals is popular in some regions because it does not have girders below a slab level. The critical part of the flat slab system is the vertical shear transfer between the slab and a column. The shear failure at the connection leads to "the pan-cake collapse" of the building, leaving no space between the adjacent floors after the collapse. Serious failure was observed in the 1985 Mexico City earthquake.

(4) Bond splitting failure

The bond stresses acting on deformed bars cause ring tension to the surrounding concrete. High flexural bond stresses may exist in members with steep moment gradients along their lengths. If the longitudinal reinforcement of a beam or column is not supported by closely spaced stirrups or ties, splitting cracks may develop along the longitudinal reinforcement, especially when the strength of concrete is low, when large diameter longitudinal bars with high strength are used, or when the concrete cover on the deformed bars is thin. These splitting cracks result in loss of bond stress, limiting the flexural and/or shear resistance at a small deformation

(5) Splice failure of longitudinal reinforcement

Longitudinal reinforcement is spliced in various ways, including lap splices, mechanical splices and welded splices. Splices should be located in a region where tensile stress is low. Splices in older buildings were located in regions of higher tensile stresses because the implications for earthquake performance were inadequately understood. Splice failure reduces flexural resistance of the member often before yielding.

(6) Anchorage failure

The force in the longitudinal reinforcement in beams and columns must be anchored within a beam-column connection or foundation. Connections of older building construction may be without joint transverse reinforcement, in which case the column and beam reinforcement is anchored in essentially plain concrete. If the beam longitudinal reinforcement is not fully anchored in a beam-column joint, the bar may pull out from the joint; e.g., beam bottom reinforcement, in non-seismic design, is embedded a short distance into the beam-column joint.

(7) Beam-column joint failure

When a moment-resisting frame is designed for weak-beam strong-column behavior, the beam-column joint may be heavily stressed after beam yielding and diagonal cracking may be formed in the connection.

Wide flexural cracks may be developed at the beam end, partially attributable to the slip of beam reinforcement within the connection. Such shear cracking may reduce the stiffness of a building. Failure is observed in beam-column joints with narrow columns and also in beam-column joints without lateral reinforcement.

(8) Failure of piles

The inertia force acting in a building must be resisted by the foundation structure. High bending moments combined with axial forces acting at the top of a pile can cause crushing of concrete. Such damage in the foundation structure is difficult to identify after an earthquake, unless apparent inclination of a building is detected as a result of permanent foundation deformation.

8.3 Quality of workmanship and materials

The performance of a construction is affected by the quality of work during construction. For example, the material strength specified in design documents may not be developed during construction. The amount of reinforcement is not placed as specified in design. The end of lateral reinforcement is not bent by 135 degrees as the building code specifies. Concrete cover to reinforcement is not sufficient and the reinforcing bar is rusted with cracks in surface concrete. Education of construction workers and inspection of construction work are necessary to maintain the quality of workmanship.

The quality of materials also deteriorates with age. Proper maintenance of structures is essential. Changes in use and occupancy often involve structural modifications without proper investigation into the consequence in the event of an earthquake.

9. Design requirements for reinforced concrete in Japan

In this part, we will review the design of reinforced concrete members in Japan. The design requirements were improved based on the lessons learned from past earthquakes.

Enforcement Regulations of the 1919 Urban Building Law specified quality of materials, allowable stresses of materials, connections, reinforcement detailing, dead and live loads, and method of calculating stresses. The 1923 Kanto Earthquake (M 7.9) caused significant damage in reinforced concrete buildings provided with (a) brick partition walls, (b) little shear walls, or constructed with (c) poor reinforcement detailing, (d) short lap splice length, (e) poor beam-column connections, (f) poor construction, or designed with (g) irregular configuration, and (h) poor foundation. The Enforcement Regulations required (a) minimum splice length of 25 times the bar diameter for lap splice, (b) use of top and bottom reinforcement in girders, (c) minimum dimensions of 1/15 times clear height for columns, and (d) minimum longitudinal reinforcement ratio of 1/80 for columns.

The Enforcement Order of the 1950 Building Standard Law specified the following for reinforced concrete construction: (a) ends of longitudinal reinforcing bars should be hooked; (b) specified compressive strength of concrete should be not less than 90 kgf/cm^2; (c) columns should be reinforced by at least four longitudinal bars firmly fastened by tie reinforcement at intervals not exceeding 30 cm and 15 times the smallest diameter of longitudinal reinforcement; (d) minimum dimension of a column section should be larger than 1/15 the clear height; the reinforcement ratio of a column was not less than 0.8 percent; (e) beams should be reinforced by top and bottom reinforcement; spacing of stirrups should be not more than 3/4 of the beam depth and 30 cm; and (f) thickness of a structural wall should be no less than 12 cm; the spacing of horizontal and vertical reinforcement should be 30 cm or less; an opening should be reinforced with bars of 12 mm diameter or larger. Two levels of allowable stress were specified for the long-term and short-term loads. The allowable stresses for long-term loading were two-thirds the specified strength for reinforcement in tension and one-third the specified compressive strength for concrete in compression.

Structural calculation for reinforced concrete construction was not specified in the Building Standard Law and associated regulations, but it was left for the individual engineer to resolve as an engineering issue. The Architectural Institute of Japan (AIJ) Standard provided the engineering basis. The 1947 AIJ standard required that (a) if the design shear stress of concrete by long-term or short-term loading exceeded the concrete allowable stress for the short-term loading, all design shear stress had to be carried by the shear reinforcement, (b) if the design shear stress exceeded $F/12$ (F: specified concrete strength) under long-term loading or $F/8$ under short-term loading, the member section had to be increased. It was generally recommended that member dimensions should be selected large enough for the concrete to carry most of the design shear stress and that the minimum amount of lateral reinforcement should be placed to ease concrete work.

The 1968 Tokachi-oki Earthquake (M 7.9) caused significant damage to short reinforced concrete columns in school buildings. The damage raised doubts about the earthquake resistance of reinforced concrete construction. The causes of the damage were summarized as (a) poor concrete and reinforcement work, (b) uneven settlement of foundation, (c) shear strength and ductility of columns, (d) corner columns under bi-directional response, and (e) torsional response of buildings. The AIJ recommended that (a) shear stress level in columns be kept low through the use of structural walls and the use of larger sections, (b) monolithic non-structural wall be included in structural analysis, (c) amount of shear reinforcement be increased and placed effectively, and (d) ends of ties and hoops be bent more than 135 degrees,

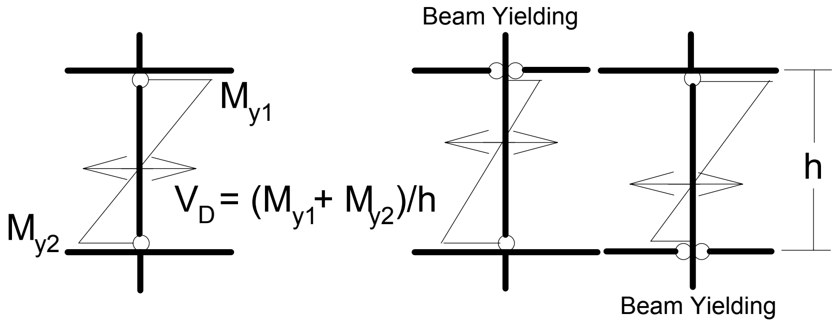

Fig. 9 Calculation of column design shear in 1971 AIJ RC Standard.

or welded closed-shape hoops and spiral reinforcement be used. Note that the shear resisting mechanism of a reinforced concrete member was not understood at the time.

The Law Enforcement Order was revised in 1971 as an emergency measures to prevent shear failure of columns; i.e., (a) diameter of hoops was set at 6 mm or larger, and (b) spacing had to be 15 cm or less (10 cm or less within a range twice the smallest dimension of column section above and below the face of horizontal members) and 15 times or less of the diameter of the smallest longitudinal bar.

The AIJ revised the RC Standard in May 1971 to its present form. The allowable shear resistance of beams and columns was derived on the basis of statistical analysis of experimental data on ultimate shear resistance. Design shear force V_D of a column may be calculated by one of the following procedures (**Fig. 9**); i.e., (a) shear force at the simultaneous flexural yielding at the top and bottom of the column, (b) shear force calculated by assuming flexural yielding at a column end and flexural yielding at the beam ends connected to the other end of the column, or (c) 1.5 times column shear under the design loads and forces. Note that the shear design is based on the capacity design concept in determining shear resistance as well as design shear force by this revision. The size of hoops and stirrups should be not smaller than 9 mm in diameter. Spacing should be not less than 10 cm; however, the spacing could be increased to 15 cm in a region 1.5 times the maximum section dimension away from the column top and bottom ends. The spacing could be relaxed to 20 cm if larger bars were used for shear reinforcement. The minimum shear reinforcement ratio was 0.2 percent. In a column where shear force was expected to increase during an earthquake, the use of welded closed-shape ties and hoops was recommended.

10. Building damage in 1995 Kobe earthquake disaster

The 1995 Hyogo-ken Nanbu Earthquake, commonly known as the Kobe Earthquake, hit a populated area of Kobe City, killing more than 5,500, collapsing approximately 92,800 buildings and houses, and damaging approximately 192,700 buildings and houses. Approximately 90 percent of the deaths were caused by the collapse of houses and buildings.

The damage to reinforced concrete buildings may be characterized by (a) collapse in a middle story in office buildings, (b) collapse in the first story in apartment and condominium buildings, (c) significant loss incurred by the damage of non-structural members, (d) fracture at the splice of longitudinal reinforcement by gas-pressured welding technique, (e) damage in lightly reinforced beam-to-column connections, and (f) failure of foundation and piles.

10.1 Damage statistics of new construction

Many reinforced concrete buildings collapsed during the 1995 Kobe earthquake due to brittle shear failure of columns. The same failure mode was observed in school buildings after the 1968 Tokachi-oki earthquake in Japan. The Building Standard Law of Japan was revised in 1971 to require close spacing of lateral reinforcement (ties) in columns. The Building Standard Law was further revised in 1981 to require higher lateral resistance from a building irregular in the distribution of stiffness in plan or along height in addition to the examination of lateral load resistance of each story at the formation of the yielding mechanism under earthquake loading; the required level of lateral load resistance was varied in accordance with the expected deformation capacity of yielding members.

The Architectural Institute of Japan investigated the damage level of all buildings in Nada and Higashi-Nada districts in Kobe City where the seismic intensity was highest; 3,911 buildings in total were investigated (Concrete Structures Committee, 1996). Seventy-five percent were residential buildings (including those used partially as offices or shops) in the area. Forty-eight percent were built in conformance with the 1981 Building Standard Law.

The damage level was classified as operational damage (no damage, light damage and minor damage), heavy damage (intermediate damage and major damage), and collapse (including those already removed at the time of investigation). Buildings with operational dam-

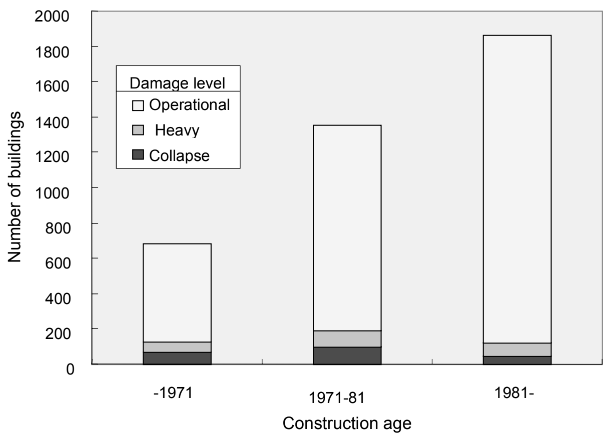

Fig. 10 Damage of reinforced concrete buildings with construction age.

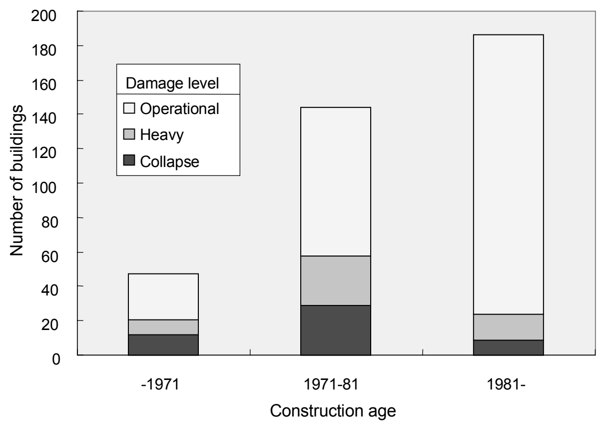

Fig. 11 Damage of soft first-story buildings with construction age.

age could be occupied immediately after the earthquake. Buildings with heavy damage needed some or major repair work for occupancy to resume.

The ratio of buildings suffering heavy damage and collapse decreased with construction age (**Fig. 10**). Among the 2,035 buildings constructed before the 1981 Building Standard Law, 7.4 percent suffered heavy damage and 8.3 percent collapsed. Among the 1,859 buildings constructed using the current Building Standard Law, 3.9 percent suffered heavy damage and 2.6 percent collapsed. The 1981 Building Standard Law enhanced significantly the performance of reinforced concrete buildings against earthquake attack. Ninety three and a half percent of the reinforced concrete buildings survived this strong earthquake motion with operational damage.

We may say that the reinforced concrete building designed using the state of the art and practice is reasonably safe against earthquakes. Approximately 15 percent, or possibly 20 percent, of those buildings constructed before the 1981 Building Standard Law need strengthening in Japan for preparation against future earthquake events.

One characteristic failure of reinforced concrete buildings in Kobe was the collapse of soft (weak) first-story buildings. This type of failure was observed in many apartment and condominium buildings, where residential units are separated by reinforced concrete structural walls, which effectively resist earthquake forces without causing much deformation. The ground floor is normally used for garage or stores. Therefore, no partition walls were placed in the ground floor. In other words, the upper stories are generally strong with ample structural walls whereas the ground floor is bare against earthquake attack. Collapse took place in the ground floor in the form of shear failure of columns.

Figure 11 compares the damage of soft first-story buildings with construction age; i.e., before the 1971 revision of the Building Standard Law, between 1971 and 1981, and after the 1981 revision of the law. Almost one half of the soft first-story buildings constructed before the 1971 revision suffered severe damage or collapse. Note a significant improvement in the safety of the soft first-story buildings with the revisions of the Building Standard Law. The ratio of heavier damage of soft first-story buildings is much larger compared with that of normal buildings. We need the improvement in design of these buildings either by limiting the deformation of the first story with the use of vibration control devices or by providing first-story columns with enhanced deformation capacity.

11. Future role of earthquake engineering

During the twentieth century, earthquake engineering concentrated on the development of technology to protect human lives from earthquake disasters. The damage statistics of reinforced concrete buildings in the 1995 Kobe earthquake disaster clearly showed the reduction of heavy damage in buildings with the development of seismic design requirements. Six and a half percent of those reinforced concrete buildings designed and constructed in accordance with the state-of-the-art suffered heavy damage or collapse, but 93.5 percent of them survived with just operational damage, even in the areas that suffered the largest ground shaking. The author believes that the state-of-the-art has reached a stage capable of protecting human lives in engineered buildings. This statement is true only when the state-of-the-art in earthquake resistant technology is applied in design and construction. Those existing buildings that do not satisfy the state-of-the-art should be retrofitted to attain the same level of safety.

The Structural Engineers Association of California (SEAOC) published “Vision 2000 - A Framework for Performance Based Engineering (Vision 2000 Committee, 1995)” in 1995. Performance-based design aims to construct a building that satisfies the planned performance of a structure under a given set of loading conditions. Extensive research is needed to achieve this design methodology.

Safety in the event of major earthquakes is one performance objective. The importance of ductility has been emphasized for the survival of a building; i.e., a structure should be capable of resistance after developing plastic deformation (damage). At the same time, ductility was used as a means to reduce design seismic forces. The author is concerned that damage may develop in a structure even during frequent minor earthquake motions because the structure is designed for too low lateral load resistance relying on large ductility. It is costly to repair structural as well as non-structural damage after minor but more frequent earthquakes, and the building cannot be used during the repair period. A structural engineer should advise a building owner about the possible cost of repairs and losses associated with having to cease building operation during repair work if a building is designed with low lateral resistance.

The damage level of structural and non-structural elements is known to be closely related to story drift (inter-story deformation). Structural damage to a brittle but high resistance building is much smaller under more frequent earthquake motions than damage to a ductile structure. A number of damage investigations reported the effectiveness of structural walls in reducing the damage in structural members as well as non-structural elements. The importance of limiting story drift during an earthquake by providing large stiffness and high lateral resistance should be emphasized in earthquake engineering.

The non-structural elements, such as windows, partitions, doors and architectural facilities, are essential parts of a building's functions. Even if structural members suffer no or slight damage, if partitions are broken in a residential building, the unit may not be occupiable until such damage is repaired or replaced. If the computer facilities are damaged in the computer and information center of a company, even though the building is free of structural damage, the function of the building is lost. The cost of repair and recovery work is often governed by the replacement of the non-structural elements rather than repair work on structural elements.

Falling of broken non-structural elements is dangerous for people escaping from the building, and falling or overturned objects may block evacuation routes in a building. The non-structural elements must be protected from minor frequent earthquakes to reduce the financial burden on the building owner as well as to maintain the function of the building. Controlling inter-story drift through the use of structural walls or structural control devices and improving the method to fasten non-structural elements to the structure may reduce damage to partitions. Stiff, weak and brittle brick walls, filled in a flexible moment-resisting frame, fail at an early stage even during medium-intensity earthquakes; such damage may be reduced by providing some gap between the brick wall and columns.

The response (acceleration or velocity) of a structure must be controlled to prevent heavy furniture and equipment from overturning on the floor or to prevent heavy equipment from falling off shelves; otherwise the contents of a building should be properly fastened to the structure.

Earthquake resistant design technology has progressed significantly in the last few decades. Damage investigations have demonstrated the poor performance of older buildings designed using out-dated technology. The retrofitting of deficient buildings is an urgent task for owners, who are responsible for maintaining the performance of their buildings to the existing code level. An efficient and reliable seismic assessment procedure should be employed to identify probably deficient buildings. New structural walls may be added to enhance the lateral resistance of weak buildings as long as the foundation has sufficient capacity to support the additional weight caused by the walls. Steel bracings can be installed if the foundation defective. The ductility of columns can be improved by steel plate jacketing or carbon-fiber plastic sheet wrapping.

12. Summary

This paper briefly reviews the development of earthquake resistant design of buildings. Measurement of ground acceleration started in the 1930s, and response calculation was made possible in the 1940s. The design response spectra were formulated in the late 1950s to 1960s. The development of digital computers made it possible to calculate the nonlinear response of simple systems in the late 1950s. Nonlinear response was introduced in seismic design in the 1960s and the capacity design concept was introduced in the 1970s for collapse safety. Earthquake engineering struggled to develop methodology to protect human lives in buildings throughout the twentieth century. The damage statistics of reinforced concrete buildings in the 1995 Kobe disaster demonstrated the improvement in building performance resulting from the development of design methodology. Buildings designed and constructed using out-dated methodology should be upgraded. The author believes that the state-of-the-art has reached a stage capable of protecting human lives in engineered buildings. Those existing buildings that do not satisfy the state-of-the-art should be retrofitted to attain the same level of safety.

The significance of the performance-based engineering should be emphasized; a building should satisfy the planned performances of a structure corresponding to a given set of loading conditions. Damage control and maintenance of building functions after an earthquake will become a major issue in the future. For this purpose, new materials, structures and construction technology should be utilized.

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