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JAPANESE SEISMIC DESIGN OF HIGH-RISE REINFORCED CONCRETE BUILDINGS - AN EXAMPLE OF PERFORMANCE-BASED DESIGN CODE AND STATE OF PRACTICES -

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SUMMARY

This paper briefly reviews the development of seismic design requirements and the construction of high-rise reinforced concrete buildings in Japan. The Urban Building Law limited the building height to 100 feet in 1919. The 1963 revision of Building Standard Law removed the height limitation, but the law required that the design and construction of high-rise buildings should be approved by the Minister of Construction because of their importance in the society and also because of the severe damage of high-rise buildings in the 1923 Kanto (Tokyo) Earthquake Disaster. The high-rise building of reinforced concrete was realized in the mid 1970s with the demand for high-quality condominium and apartment buildings in urban areas. Performance-based design regulations were introduced in the 1998 revision of Building Standard Law. A separate notification was issued to define performance requirements for high-rise buildings, but no design calculation methods were specified. This paper presents the state of practices to satisfy the performance-based regulations for gravity loads, snow loads, wind forces and earthquake forces with emphasis on the design of reinforced concrete structures.

INTRODUCTION

Development of Building Code in Japan

The construction of reinforced concrete buildings in Japan was promoted by Professor Toshikata (Riki) Sano (1880-1956) of the University of Tokyo after he investigated the building damage of the 1906 San Francisco earthquake disaster. He noted the good performance of reinforced concrete buildings against fire and also earthquake shaking.

The Urban Building Law and Urban Planning Law were promulgated in 1919 to regulate buildings and city planning in six major cities (Tokyo, Yokohama, Nagoya, Kyoto, Osaka, and Kobe) at the time. The former was the first building code in Japan. The Urban Building Law Enforcement Order, issued in 1920, limited the building height to 65 feet in residential areas and to 100 feet in non-residential areas to maintain a uniform urban scene. The Building Law Enforcement Regulations, issued in 1920, specified

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structural design requirements for timber, masonry, brick, reinforced concrete and steel construction; the quality of materials, connections, reinforcement detailing, dead and live loads, and method of calculating stresses in section were prescribed in the regulations, but design earthquake and wind forces were not specified.

The 1923 Kanto (Tokyo) Earthquake (M 7.9) caused significant building damage in Tokyo and Yokohama areas, with a loss of life more than 140,000, heavy damage in more than 250,000 houses, fire loss of more than 450,000 houses. Some significant damage was observed in steel and reinforced concrete construction. More than 90 percent of the dead and lost were caused by fire in Tokyo. The damage to reinforced concrete buildings was relatively low in Tokyo City although these buildings were not designed to resist earthquake motions. Fifteen reinforced concrete buildings totally collapsed, 20 half collapsed, 49 severely damaged, 74 lightly damaged, but 515 undamaged; i.e., twenty percent of reinforced concrete buildings suffered some damage, but 80 percent of reinforced concrete buildings survived with minor or no damage. The damage was observed in buildings having (a) brick partition walls, (b) little structural walls, (c) poor reinforcement detailing, (d) short lap splice length of longitudinal reinforcement, (e) poor beam-column connections, (f) poor construction quality, (g) irregular configuration, and (h) poor foundation.

The 1924 revision of Urban Building Law Enforcement Regulations introduced the use of design seismic coefficient of 0.10. The use of seismic coefficients was proposed by Professor Toshikata Sano in his doctoral thesis in 1916. He assumed that a building was rigid and fixed on the ground; the building, therefore, was subjected to the horizontal acceleration of the ground without dynamic amplification due to the flexibility of the structure. Maximum ground acceleration in downtown Tokyo was estimated to be 0.3 G during the 1923 Kanto Earthquake, in which G is gravitational acceleration. This peak ground acceleration was reduced, for design purpose, to 0.10 by considering the safety factor of 3 used in determining the allowable stress level relative to the material strength. The height of reinforced concrete buildings was implicitly limited to 20 m by the administrative guidance for many years.

The application of the Urban Building Law was gradually extended to the whole country. After the World War II, the Building Standard Law (1950) replaced the role of the Urban Building Law regulating the building construction throughout the country. The building height limitation of 100 feet was maintained in the Building Standard Law Enforcement Order (1950) due to the bitter experience of the 1923 Kanto Earthquake Disaster. The design seismic coefficient was increased to 0.2 because the allowable stress of materials was also doubled for seismic loading in the Building Standard Law Enforcement Order.

Construction of High-rise Buildings

The Building Standard Law Enforcement Order, revised in 1963, removed the building height limitation. With this revision, the design and construction of buildings taller than 45 m was required the approval of the Minister of Construction. The Minister of Construction deputed the task of technical review to a technical appraisal committee formed in the Building Center of Japan. The Hotel New Otani, the first high-rise steel building taller than 100 feet, was constructed to accommodate visitors to the 1964 Tokyo Olympic Games. The Building Standard Law Enforcement Order, revised in 1981, relaxed the height limitation to 60 m, necessary for the approval of the Minister of Construction.

The construction of high-rise buildings in a seismic country was made possible by the following technical development in the late 1950s and early 1960s; i.e., (a) the observation of strong earthquake ground motions, (b) the understanding of the behavior of structural members under reversal loading to failure in the laboratory, and (c) the enhanced use of digital computers in the static analysis of framed buildings and the earthquake response analysis of lumped mass-spring systems.

The 36-story Kasumigaseki Building, completed in April 1968, was the first so-called skyscraper (147 m high) in Japan; the structural design was supervised by Dr. Kiyoshi Muto (1903-1989), Professor Emeritus of University of Tokyo and vice-president of Kajima Corporation. Most high-rise buildings were designed and constructed using steel because the steel is strong and ductile material and easy to control material properties and fabrication accuracy in the factory. The construction of reinforced concrete high-rise buildings was much delayed due to the concern about the lack of deformation capacity.

The 1968 Tokachi-oki Earthquake revealed the weakness of reinforced concrete buildings; i.e., shear failure of reinforced concrete columns was observed in school buildings. An integrated national research program was organized by the Ministry of Construction to clarify the cause of reinforced concrete failure and prevent similar failure in the future reinforced concrete construction. Extensive laboratory tests of reinforced concrete members were carried out in the laboratories of universities and industrial as well as government research institutes throughout the country. The shear design of reinforced concrete members was improved through this investigation. This study also provided the knowledge to enhance the ductility of reinforced concrete members, in general, by preventing brittle modes of failure.

Construction of High-rise Reinforced Concrete Buildings

There was a demand to construct high-rise condominium and apartment buildings in the early 1970s to solve the housing shortage in urban areas as well as to improve the housing quality. The reinforced concrete building is suited for residential purpose because of superior properties of reinforced concrete construction such as; (a) sound insulation between adjacent floors and units, (b) heat insulation capacity, (c) little vibration under strong winds, (d) fire resistance, (e) durability, and (f) competitive construction cost against steel.

Kajima Corporation, under the guidance of Dr. Kiyoshi Muto, investigated the method to improve the ductility of reinforced concrete members by lateral confining reinforcement, the method of nonlinear earthquake response analysis and the method for efficient construction and strict quality control. An 18-story apartment building (47.7 m in height) for the employees of Kajima Corporation was the first high-rise reinforced concrete construction, completed in 1974. The structure was moment-resisting frames with 3.0-m span in the longitudinal direction and 4.5-m span in the transverse direction. Other major construction companies followed the Kajima's efforts.

A technical review committee, chaired by Professor Hiroyuki Aoyama of University of Toyo, was formed at the Building Center of Japan in 1984 to examine and discuss the structural design procedure and construction technology for the rational development of high-rise reinforced concrete construction; working groups were formed to discuss technical development for each construction company, separately. Different structural design criteria were fostered in each company on the basis of its own experimental evidences of structural members under load reversals at their own research laboratory, and the response calculated by computer programs for nonlinear static as well as dynamic analysis. Construction technology for production of high-strength concrete, efficient and accurate fabrication of reinforcement cages, election of reinforcement cages in formwork and placement of concrete was also developed in each construction company, separately.

The construction of high-rise reinforced concrete buildings (more than 45 m high before 1981, and 60 m high after 1981) in Japan is summarized in Fig. 1. Before and in 1985, the total number of reinforced concrete and steel-encased reinforced concrete (SRC) composite buildings was slightly more than 50, but most were constructed using the SRC composite construction. Concrete filled tube (CFT) column systems replaced the SRC construction in recent years due to the advantage in construction cost and efficiency. The construction of high-rise reinforced concrete buildings increased steadily to date.

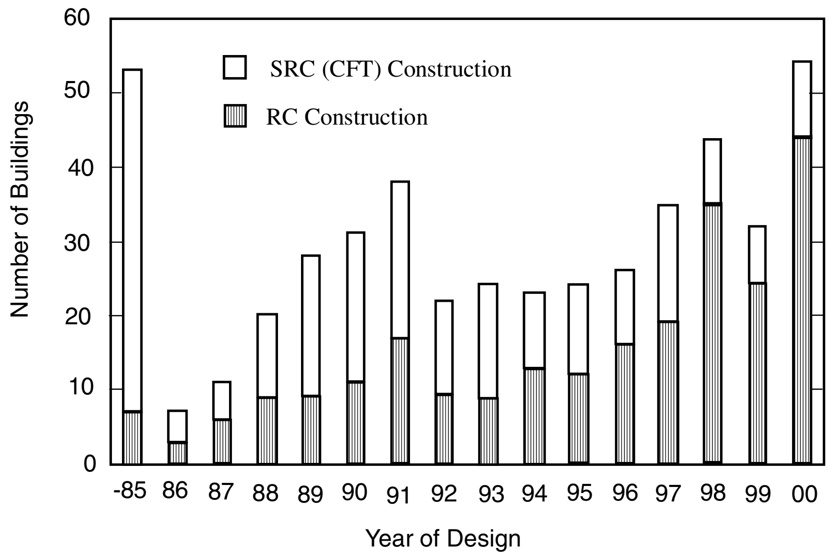


Fig. 1: Construction of High-rise Reinforced Concrete Buildings in Japan (Izumi [2])

The Ministry of Construction launched a five-year integrated national research program (1988-1993), entitled "Development of Advanced Reinforced Concrete Buildings using High-strength Concrete and Reinforcement (generally called New RC program)," with enthusiastic participation of construction companies. University researchers were invited to carry out experimental and analytical work to support the research program. The objectives were (a) to develop technology for production and quality control of high strength concrete (60 to 120 MPa) and steel reinforcement (800 to 1,200 MPa), (b) to investigate the properties of high strength materials, (c) to investigate the performance of members using high-strength materials under lateral load reversals, (d) to develop structural design specifications, and (e) to develop the construction specification using high strength materials in high-rise buildings. The research results enhanced the design and construction capability of high-rise reinforced concrete buildings in Japan. The range of concrete strength used in high-rise buildings is shown in Fig. 2. With the progress in research of the New RC program, high strength concrete was gradually adopted in the construction. The New RC project is presented in a book edited by Aoyama [1].

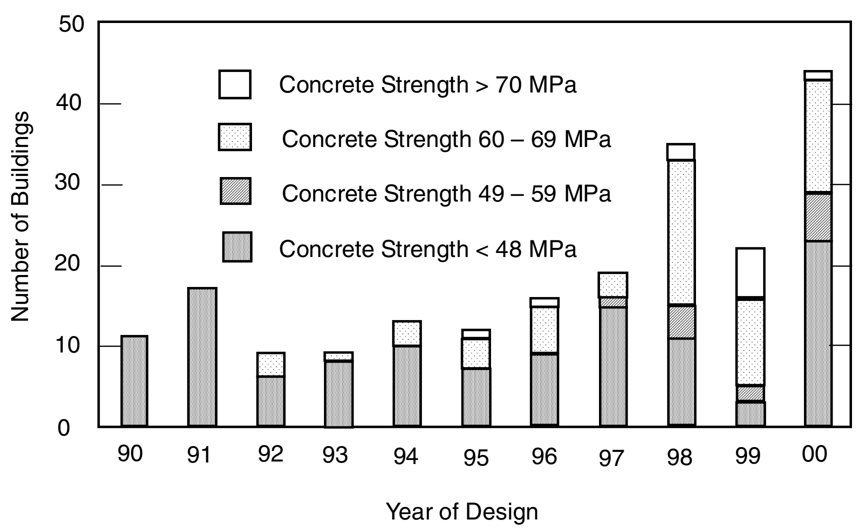


Fig. 2: Use of High-strength Concrete in High-rise Construction (Izumi [2])

Various new technical developments were gradually introduced in the structural design and construction of high-rise reinforced concrete buildings; for example, (a) the use of high-strength materials after the New RC program (1988-1993), (b) precast concrete elements in large scale from the latter part of the 1990s, (c) new structural systems (tube structures, core walls) from the latter part of the 1990s, (d) passive vibration control devices (hysteretic or visco-elastic energy dissipating devices) in the mid-1990s (Fig. 3) and (e) the use of base-isolation systems from the end of the 1990s. Longer spans were made possible in reinforced concrete buildings by the use of high-strength materials, the adoption of new structural systems and the use of wide and shallow girders. The development in reinforced concrete technology was welcomed by the developer and architect because freer architectural planning became possible using reinforced concrete. The use of precast concrete elements was effective improving the quality of products, improving construction methods and reducing the construction time. The high strength concrete is more expensive, but the high strength can reduce the volume of concrete in the structure and improve the durability. One of the drawbacks in using high strength concrete was explosive spalling of shell concrete under fire. This phenomenon was observed more in structural members using higher strength concrete, but the problem was solved by mixing plastic fiber in the concrete.

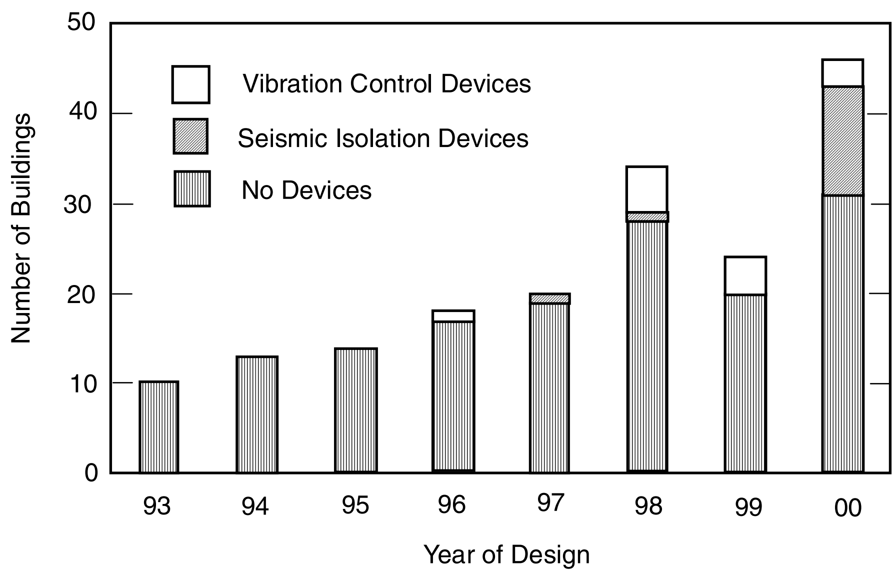


Fig. 3: Use of Vibration Control Devices in High-rise Reinforced Concrete Construction (Izumi [2])

It should be noted that vibration control devices become generally effective only after the building deforms to a certain extent. The reinforced concrete building was previously believed to suffer significant damage before the vibration control devices could start to function during an earthquake. The use of high-strength materials reduced the section dimensions and associated member stiffness and also increased the deformation of members at elastic limit. The use of long spans increased the flexibility of a high-rise building. Therefore, the vibration control devices have been proven to be effective in reducing earthquake response in high-rise reinforced concrete construction. The vibration control devices were introduced in high-rise reinforced concrete construction in 1996 and the seismic isolation devices were introduced in 1997. The number of their applications became significant in 1998; the vibration control or seismic isolation devices are used in almost one-third of the high-rise reinforced concrete buildings in 2000.

In 2004, a 56-story reinforced concrete residential building is to complete in Tokyo. The concrete strength in the lower story columns was 100 MPa.

CURRENT BUILDING CODE SYSTM IN JAPAN

Building Standard Law

The Building Standard Law of Japan was proclaimed as a national law in May 1950. The objectives were to "safeguard the life, health, and property of people by providing minimum standards concerning the site, structure, equipment, and use of buildings." The structural requirements are outlined in Article 20 that "the building shall be constructed safe against dead and live loads, snow loads, wind forces, soil and water pressures, and earthquake and other vibration forces and impacts." The article also requires that the construction and structural calculation shall conform to the technical standard outlined by a cabinet order, Building Standard Law Enforcement Order, issued for the technical enforcement of the Building Standard Law. The law requires that design documents and drawings should be submitted to a municipal government which should examine the design documents and confirm that the design and construction satisfy the legal provisions. This requirement made the code prescriptive because building officials must be able to judge the legal conformity to the regulations. The construction should be inspected by a building official in the municipal government after the completion of the work. The number of building officials in local government was not sufficient to carry out their duty in comparison with the number of applications for building construction.

The framework of Building Standard Law was significantly revised in 1998,

(1) Introducing performance-based regulations wherever feasible,
(2) Allowing private agencies to execute the building confirmation and construction inspection works during and after the construction,
(3) Deregulating urban land use, and
(4) Allowing public access to design and inspection documents.

The performance-based regulation is intended (a) to specify the requirements by performance, and (b) to relax and remove administrative control. Such performance-based requirements in building codes are generally encouraged in recent years with the expectation to expand the scope of structural design, especially for the application of new materials, construction technology and structural systems. It is further expected to remove international trade barriers in the construction markets and to encourage the engineer to develop and apply new construction technology and engineering.

Building Standard Law Enforcement Order

The Building Standard Law Enforcement Order is issued by the cabinet to provide technical requirements for the law. The construction and structural calculation requirements are specified in Chapter 3 "Structural Strength." Specification requirements about construction are outlined in Sections 1 through 7, including mandatory specification requirements associated with (a) basis of structural calculations, (b) quality of construction materials, (c) durability of structural members, (d) workmanship during construction and (e) safety against fire; these specification requirements are called "durability related provisions". The durability provisions cannot be replaced by the examination of structural performance through structural calculation. Structural calculation methods are outlined in Section 8 for (a) the allowable stress calculation (old procedure) and (b) the ultimate strength calculation (new procedure).

Building Standard Law Enforcement Order was revised in 2000 to enforce the 1998 revision of the law. Significant revisions were made toward performance-based requirements in the area of fire protection and evacuation. However, relatively small revisions were made in structural design requirements because the structural design regulations were already in a performance-based format. The capacity-demand spectrum method was introduced in seismic design in the 2000 revision.

Three performance objectives were defined for the evaluation and verification of performance (response) under (a) gravity loads, (b) snow loads, (c) wind pressures, and (d) earthquake forces; i.e.,

(1) Maintenance of building serviceability under permanent loading conditions (dead and live loads),

(2) Prevention of structural damage under frequent loading conditions (snow, wind and earthquake events corresponding to a return period of approximately 50 years), and

(3) Protection of occupants' life under extraordinary loading conditions (snow, wind and earthquake events corresponding to a return period of approximately 500 years.

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.

The Building Standard Law Enforcement Order requires that the construction and structural calculation shall follow one of the following routes;

Route 1: Construction shall conform to the specification provisions. The structural calculation shall conform to the allowable stress calculation or one of the calculation methods outlined by the Minister of Land, Infrastructure and Transport (the Ministry of Construction was reorganized to the Ministry of Land, Infrastructure and Transport in 2001, and hereafter abbreviated as "MOLIT") as a procedure equivalent or superior to the allowable stress calculation.

Route 2: Construction shall conform to the durability related provisions of the specification provisions. The structural calculation shall conform to the ultimate strength calculation or one of the calculation methods outlined by MOLIT as a procedure deemed to ensure a safety level of a building equivalent or superior to the ultimate strength calculation.

Route 3: Construction shall conform to the durability related provisions. The structural calculation shall conform to one of the structural calculation methods, outlined by MOLIT.

Construction and structural calculation methods outlined by MOLIT are issued to supplement the Building Standard Law Enforcement Order in the form of Notifications of the Minister.

The Building Standard Law requires that the structural method of buildings exceeding 60 m in height shall follow Route 3 above. The safety of the building shall be examined by studying the stress and deformation of structural members under combined design loads and forces taking into account the dynamic characteristics and construction of a building.

MOLIT Notifications

Numerous notifications have been issued by MOLIT to outline the detailed technical requirements or to specify the calculation methods stated in the Building Standard Law Enforcement Order; for example;

- Notification 1918 of year 1987: Specification of seismic zone factor Z of regions, calculation of dynamic characteristic factor R_t and story shear distribution factor A_i (Article 88).
- Notification 1348 of year 2000: Structures of roofing elements, exterior finishing elements and exterior curtain walls (Article 39).
- Notification 1454 of year 2000: Calculation of values E (environmental factor to influence wind velocity) and V_o (design wind velocity of regions) and wind force coefficient (Article 87)
- Notification 1455 of year 2000: Criteria to determine heavy snow areas and to define the snow depth for snow loads (Article 86).
- Notification 1457 of year 2000: Calculation of parameters necessary for the ultimate strength calculation (Demand- and capacity-spectra procedure) (Article 82).
- Notification 1458 of year 2000: Structural calculation for the safety of roofing elements, exterior finishing elements and exterior curtain walls under wind pressure (Article 82).

- Notification 1459 of year 2000: Criteria to judge interference of serviceability of buildings in structural calculation (Article 82).
- Notification 1462 of year 2000: Strength of concrete in construction to ensure the design concrete strength (Article 74).
- Notification 1463 of year 2000: Structural requirements for reinforcement splices in reinforced concrete construction (Article 73).

BUILDING CODE REQUIREMENTS OF HIGH-RISE BUILDINGS

The Building Standard Law Enforcement Order (Article 36) requires that "the structural design and construction of high-rise buildings shall satisfy the durability related provisions, and shall be approved by the Minister of Land, Infrastructure and Transport (MOLIT) to be structurally safe based on the calculation procedure outlined by MOLIT." The high-rise building is defined as a building taller than 60 m in the order. The Building Standard Law Enforcement Order (Article 81) also requires that "the structural calculation of high-rise buildings shall follow the calculation procedure, outlined by MOLIT, which can verify the structural safety of the building by evaluating the local action and deformation of the structure continuously taking into consideration the construction methods and dynamic characteristics of the structure." This article requires the dynamic analysis of the structure under earthquake motions.

Notification No. 1461 of year 2000 was issued by MOLIT to outline the structural calculation standard to verify the structural safety of high-rise buildings (Article 81 of the Building Standard Law Enforcement Order). Followings are not official translation, and the reader is advised to refer to the original Japanese documents. The notification is written in a performance-based format consisting of eight articles;

Article 1: No structural members shall be damaged under the dead and live loads representing actual conditions, and other loads and forces acting on all parts of the building.

Article 2: Following structural calculation shall be made for snow loads on the building. The snow load can be reduced for the structure where the snow melting devices are installed or special measures are taken to reduce snow loads.

(a) Snow load shall be determined in accordance with Article 86 of the Building Standard Law Enforcement Order. If an expected value associated with a 50-year return period is assessed for the construction site by special study or investigation, the assessed value can be used.
(b) No structural members shall be damaged under snow load defined in (a).
(c) The structure shall not collapse under the snow load equal to 1.4 times the value defined in (a).

Article 3: Following structural calculation shall be made for wind forces acting on the building. The effect of vibration normal to the wind direction in the horizontal plane and torsional vibration on structural response, and the effect of vertical vibration on the roof elements shall be appropriately taken into consideration in the structural calculation.

(a) No structural members shall be damaged under rare strong winds which produce a wind velocity equal to or higher than the average wind velocity at 10 m above ground level taking into consideration ground roughness defined by Article 87 of the Building Standard Law Enforcement Order. This requirement does not apply to vibration control devices whose fatigue, hysteresis and damping characteristics have been established to be effective during extremely rare winds and earthquake motions.
(b) The structure shall not collapse by extremely rare strong winds which produce an average wind velocity 1.25 times the value defined in (a) at 10 m above ground level.

Article 4: Following structural calculation shall be made for earthquake forces acting on the building. The effect of vertical ground motion considering the size and configuration of the building, the effect of ground motion normal to the principal ground motion concerned, the effect of phase difference of ground motion, and the effect of vertical loads under horizontal sway shall be appropriately taken into consideration in the structural calculation.

(a) The ground motion acting on structures in the horizontal direction is defined in parts 1) to 4) below. If the ground motion is determined taking into consideration the effect of faults in the vicinity of the construction site, the effect of epicentral distance and other characteristics of seismic motions and the influence on structural response, the followings may not be satisfied.

1) Acceleration response spectrum (a curve representing acceleration response characteristics of structural systems with respect to their period at a 5 % damping factor) of the ground motion on the open engineering bedrock (the engineering bedrock is a soil layer located below the structure with sufficient thickness and rigidity having a shear wave velocity larger than 400 m/sec, and the open engineering bedrock is a bedrock free from the effect of surface soil layers above) shall satisfy the values given in Table 1, and the amplification of ground motion by surface geology should be considered in defining the design ground motion.

Table 1: Design Acceleration Spectrum at Open Engineering Bedrock

Period, sec	Acceleration response spectral value, m/sec^2	
	Rare earthquake ground motion	Extremely rare earthquake ground motion
$T<0.16$	$(0.64+6T)Z$	Five times the acceleration response values defined for the rare earthquake ground motion
$0.16<T<0.64$	$1.6Z$	
$0.64<T$	$(1.024/T)Z$	
T: period of structure, sec. Z: seismic zone factor defined in Article 88, Part 1 of Building Standard Law Enforcement Order.		

2) The duration of motion shall be longer than 60 sec.

3) The earthquake ground motion (acceleration, velocity or displacement or their combination) shall be digitally defined at appropriate time intervals.

4) The number of ground motions shall be large enough to verify the safety of the structure under the effect of earthquake motion.

(b) Structural members shall be examined not to be damaged under the rare earthquake ground motions defined in (a) using the equation of motion. Structural vibration control members are exempted from this requirement.

(c) The structure shall be examined not to collapse under the extremely rare earthquake ground motions defined in (a) using the equation of motion.

Article 5: Loads and forces defined in Article 1 shall be used in the structural calculation specified in Articles 2 through 4.

Article 6: The deformation and vibration of structural members under loads and forces defined in Article 1 shall not interfere with the use of the building.

Article 7: Roofing elements, exterior finishing materials and exterior curtain walls shall be structurally safe under the wind forces, earthquake forces and other impact forces.

Article 8: In a building located within a land failure warning zone, exterior walls shall not fail under the forces caused by the land failure of slope considering the types of natural hazards. The loads and forces defined in Article 1 shall be considered in the examination.

The loads and forces, amplitudes of wind forces and target acceleration spectrum for earthquake ground motions are specified, and the performance of a structure are specified. It should not noted, however, that no specific material properties, structural dimensions, calculation methods are specified in the notification. The appropriateness of structural calculation and verification of structural performance used in design calculation should be examined by specialists who have sufficient knowledge and experience in assessing earthquake ground motion, dynamic analysis, structural analysis and the behavior of structural members before the approval by MOLIT.

DESIGN FOR VERTICAL LOADS

This and subsequent sections introduce the state of practices in design and construction of high-rise buildings in accordance with Notification No. 1461 of year 2000.

Permanent Loading

The performance requirement under gravity loading is expressed in Article 1. No structural members shall be damaged under the dead and live loads representing actual conditions of the building, and other loads and forces acting on all parts of the building.

The stresses at critical sections of structural members are calculated by a linearly elastic structural analysis method under the dead and live loads. Flexural and shear deformations of structural members are considered in the structural analysis, but axial deformation is ignored; beam-column connections are assumed to be rigid. Structural members may be judged undamaged when the stresses in the member are smaller than "the allowable stresses of materials specified for the long-term loading."

For the long-term loading, the allowable compressive stress of concrete is one-third of the specified concrete strength F and allowable shear (and tensile) stress is $F/30$. The allowable stress of reinforcement for tension and compression is two-third of the specified yield stress F; but the allowable stress shall not be larger than 215 MPa for bars of nominal diameter smaller or equal to 28 mm, nor larger than 195 MPa for bars of nominal diameter larger than 28 mm. The allowable tensile stress of shear reinforcement is also $2F/3$, but shall not be larger than 195 MPa. The allowable bond stress between reinforcing bars and concrete is $F/15$ for top reinforcement in the beam and girder if the specified concrete strength F is less than or equal to 22.5 MPa, and $0.9+2F/75$ when F is larger than 22.5 MPa. The allowable bond stress along the longitudinal reinforcement in the other region is 1.5 times the allowable bond stress for the beam top reinforcement.

It should be noted that the damage of the materials is minimal and the linearly elastic structural analysis can be justified as long as the stress is less than the allowable stresses of the materials for the long term loading.

The serviceability of a building under gravity loads and forces shall be examined as required in Article 6 of the notification. Notification 1459 of year 2000 outlines the structural calculation for the examination of serviceability under gravity loading conditions; the slab depth shall be more than one-thirtieth of clear support distance in the shorter direction, and the beam depth shall be more than one-tenth of support distance unless the deformation of horizontal members, calculated as the product of elastic deformation under dead and live loads and the creep deformation amplification factor (8.0 for reinforced concrete girders and 16.0 for reinforced concrete slabs) is examined to be smaller than 1/250 of the support distance.

Snow Loading

The performance requirement under snow loading is expressed in Article 2 of the notification. The Building Standard Law Enforcement Order requires that the vertical snow depth for design shall be specified by the local jurisdiction. Criteria to specify heavy snow regions and determine the design snow depth are outlined in Notification 1455 of year 2000. The design snow depth is evaluated for the maximum snow fall of consecutive days expected over a 50-year return period (annual probability of exceedance of 0.02). The structural member shall not be damaged under the design snow loads.

Structural members are thought to be undamaged if the maximum stresses at critical sections under the combined effect of dead, live and snow loads are less than "the allowable stresses of materials for the short term loading." The allowable compressive and shear stresses of concrete for the short term loading are twice the corresponding allowable stresses for the long term loading; i.e., allowable compressive stress of concrete is two-third of the specified compressive strength and allowable shear stress is one-fifteenth of the specified compressive strength. Allowable stress of reinforcement for the short-term loading is equal to the specified yield stress, but allowable tensile stress of shear reinforcement shall be not greater than 390 MPa. The level of these allowable stresses for the short-term loading corresponds to the elastic limit of the materials. Therefore, the linearly elastic structural analysis can be justified for this design verification.

The notification also requires that the structure shall not collapse under the snow load corresponding to 1.4 times the design snow load combined with the dead and live loads. The multiplier 1.4 was determined as a ratio of the maximum snow depth expected over a 500-year return period to that expected over a 50-year return period. Collapse can be examined if a collapse mechanism might be formed under the specified loading. However, the dead load of a reinforced concrete building is large compared with live and snow loads, the maximum stresses at critical sections of structural members can be normally shown to be smaller than the allowable stresses of materials for the short term loading. Therefore, the examination of collapse is not necessary.

DESIGN FOR WIND FORCES

The performance requirement of a building under design wind forces is expressed in Article 3 of the notification. Design velocity pressure W_s (N/m^2) of winds acting normal to the building surface is defined as

$$W_s = q \cdot C_f \tag{1}$$

where C_f is wind force coefficient, specified in Notification No. 1454 of year 2000. q is design velocity pressure (N/m^2) defined in the Building Standard Law Enforcement Order;

$$q = 0.6\, E\, V_0^2 \tag{2}$$

where, E is a factor representing the influence of height at the roof level and the effect of neighboring structures and trees on the amplitudes of wind velocity, and V_0 is the basic wind velocity (m/sec) defined as the average wind velocity over 10-minute period at 10.0 m above the ground.

The basic wind velocity V_0 of rare strong wind events (Level 1 wind), corresponding to a return period of 50 years, is specified in Notification No. 1454 of year 2000 taking into consideration the past history of typhoons and wind disasters in each region. The velocity varies from 30 m/sec to 46 m/sec in the country. The factor E, also specified in Notification No. 1454, is calculated by taking into consideration the distribution profile of wind velocity along the height and the gust response factor.

Structural members shall not be damaged under Level 1 winds; i.e., the maximum stresses at critical sections under the combined effect of dead, live and Level 1 wind forces shall be less than "the allowable stresses of materials for the short term loading."

The basic wind velocity V_0 of extremely rare strong wind events (Level-2 wind), corresponding to a return period of 500 years, is 1.25 time the basic wind velocity of Level-1 winds, determined on the basis of statistical study. Therefore, the Level-2 design wind pressure is 1.56 (=1.25x1.25) times the Level-1 design wind pressure.

The structure shall not collapse under Level-2 winds. However, the maximum stresses at critical sections of structural members under the Level-2 wind forces have been shown much less than the allowable stresses of materials for the short term loading. The structural member of a Japanese reinforced concrete building is normally proportioned to satisfy the stress caused by design seismic forces.

For a building taller than 100 m, if an aspect ratio ($H/\sqrt{BD}$, where H is the height of building, B and D are the width and depth of the building in plan) is greater than 3.0, the vibration caused by wind pressure acting normal to the wind and the torsional vibration must be examined. If the following equation is satisfied, this examination can be exempted;

$$U_H < 2.5\,\eta_0\,\sqrt{BD} \qquad (3)$$

where, η_0 is the fundamental frequency of building normal to wind direction or of torsional vibration, U_H : design wind velocity (m/sec). No specific method is specified for the examination of vibration caused by wind pressure acting normal to the wind and torsional vibration in the notification.

If an aspect ration of the building of rectangular plan is larger than 4.0, the structure should be examined against possible vortex shedding and aerodynamic instability. If the following equation is satisfied, this examination can be exempted;

$$U_H \le 0.83\,U_{cr}^{*}\,\eta_0\,\sqrt{B\,D} \qquad (4)$$

where, U_{cr}^{*} is non-dimensional wind velocity to initiate the vibration toward aerodynamic instability. No specific method is specified for the examination of vortex shedding and aerodynamic instability in the notification.

Methods to examine the transverse and torsional vibration, vortex shedding and aerodynamic instability are outlined in "Recommendations for Loads on Buildings and Commentary" published by Architectural Institute of Japan [3].

Article 7 of the notification requires that roofing elements, exterior finishing materials and exterior curtain walls shall be structurally safe under the wind forces. The strength of these elements due to out-of-plane wind pressure and these fixing parts must be examined for the design wind pressure.

DESIGN FOR EARTHQUAKE FORCES

The performance of a building under earthquake motions is outlined in Article 4 of the notification, and must be examined by earthquake response calculation solving the equation of motion.

Intensity of Design Earthquake Motions

For rare earthquake events (Level 1), artificial ground motions compatible with a response acceleration spectrum, shown in Fig. 4, specified at the open engineering bedrock must be generated. The engineering bedrock is defined as a thick soil layer at which shear wave velocity is larger than 400 m/sec; the engineering bedrock is typically a hard soil layer which can support the pile foundation or mat foundation of a high-rise building.

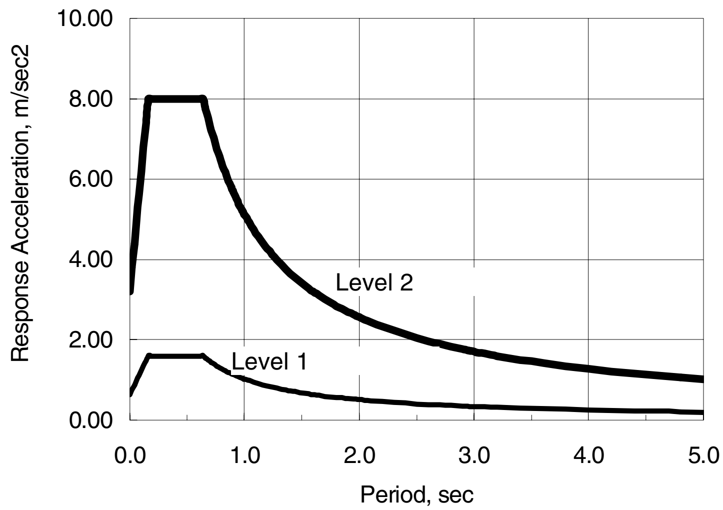


Fig.4: Response Spectrum of Design Earthquake Motions at Open Engineering Bedrock

For extremely rare earthquake events (Level 2), artificial earthquake ground motions shall be generated either (a) compatible with response acceleration spectrum five times that of Level 1 motions specified at the open engineering bed rock or (b) taking into consideration the effect of faults near the construction site, magnitude, epicentral distance and other characteristics of seismic motions.

More than two ground motion records must be prepared each for Level-1 and Level-2 earthquake events. The duration of artificially generated ground motions must be longer than 60 sec to excite a long-period structure.

It should be noted that the intensity of Levels 1 and 2 ground motions for the high-rise buildings are specified the same as those for the normal building less than 60 m in height. The intensity of Level 1 ground motions, specified in the Notification, is believed by some engineers to be too small for the damage control of such important facilities as high-rise buildings. At most design appraisal agencies, therefore, the use of additional observed earthquake records, representing the geological conditions at the construction site, is recommended in design calculation. The maximum velocity of observed ground motions is normalized to 250 mm/sec for Level-1 earthquake events, and 500 mm/sec for Level-2 events.

Generation of Spectrum Compatible Ground Motions

Earthquake ground motions compatible with a specified response spectrum can be generated in various techniques such as (a) the use of random phase angle with envelope time function and (b) the use of phase differences of Fourier spectrum of observed ground motions.

Generation of Earthquake Motions using Random Phase Angle

Acceleration time history of a ground motion is represented as the product of envelope time function $e(t)$ and Fourier function as

$$\ddot{y}(t) = e(t)\sum_{i=1}^{n} A_i \cos(\omega_i t + \phi_i) \tag{5}$$

in which A_i, ω_i, ϕ_i are Fourier amplitude, circular frequency and phase angle. The envelope function (Fig. 5) should consider the magnitude of an earthquake and the distance to the epicenter.

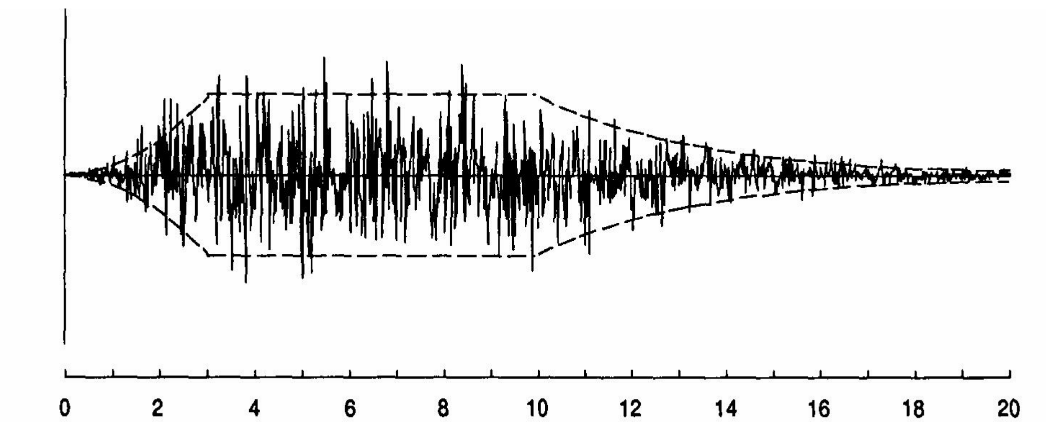


Fig. 5: Envelope Function for Generation of an Earthquake Accelerogram

The following envelope functions are often used after Jennings et al. [4]. For Level-1 earthquake events;

$$\begin{aligned} e(t) &= (\tfrac{t}{5})^2 && 0 \le t < 5 \text{ sec} \\ e(t) &= 1.0 && 5 \le t < 25 \text{ sec} \\ e(t) &= \exp\{-0.066(t-25)\} && 25 \le t < 60 \text{ sec} \end{aligned} \tag{6}$$

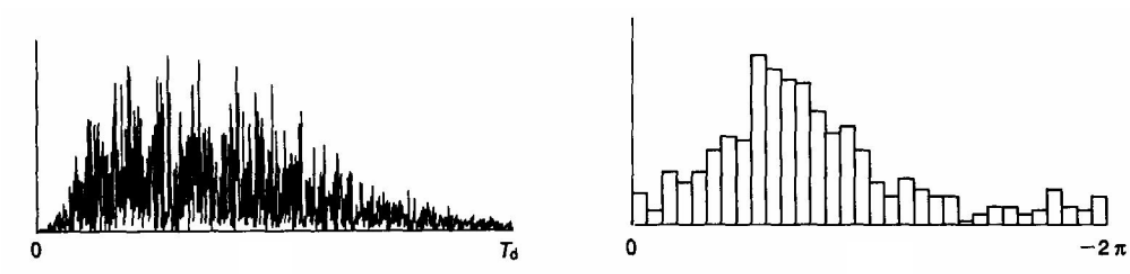
For Level-2 earthquake events;

$$\begin{aligned} e(t) &= (\tfrac{t}{5})^2 && 0 \le t < 5 \text{ sec} \\ e(t) &= 1.0 && 5 \le t < 35 \text{ sec} \\ e(t) &= \exp\{-0.027(t-35)\} && 35 \le t < 120 \text{ sec} \end{aligned} \tag{7}$$

Phase angle ϕ_i is taken as a random variable in a range of 0.0 to 2π for each circular frequency ω_i. Fourier amplitude A_i at frequency ω_i is revised iteratively until (a) the pseudo-velocity spectrum amplitudes of generated motion become more than 85 % of the target response spectrum at each frequency, (b) the covariance of error in generated response spectrum relative to the target spectrum is less than 0.05, and (c) average error becomes less than 0.02 over the entire frequency range. The initial values of Fourier amplitudes A_i may be taken from the target pseudo-velocity spectrum because velocity spectrum ordinates at zero damping are similar to Fourier spectrum amplitudes of the motion.

Generation of Earthquake Motions using Phase Differences of Observed Motions
It is empirically observed that the distribution of ground motion amplitudes over the duration resembles to the distribution of phase differences $\Delta\phi_i$ between 0.0 and 2π of the Fourier expansion of a ground motion (Ohsaki [5], Fig. 6). Therefore, the probability density function of phase angle differences is assumed similar to the envelope curve of an observed ground motion. A random number between 0.0 and 1.0 is transformed to a phase angle difference $\Delta\phi_i$ between 0.0 and 2π using an accumulated probability density function. A phase angle ϕ_i for circular frequency ω_i is determined by adding phase difference $\Delta\phi_i$ to a previous phase angle ϕ_{i-1}.

$$\ddot{y}(t) = \sum_i A_i \cos(\omega_i t + \phi_i) \tag{8}$$



(a) Amplitude Distribution of Earthquake Motion (b) Phase Angle Difference Distribution

Fig. 6: Amplitude and Phase Angle Difference Distributions of Earthquake Motion, (Ohsaki [5])

The phase angle of Fourier spectrum of an observed earthquake motion can be directly used to generate an artificial ground motion rather than using the probabilistic distribution of phase angle differences. This simplifies the work. If a near fault earthquake motion, such as the 1995 Kobe Marine Observatory record, is used as an observed record, the duration of large amplitude motion becomes shorter although the intensity of the generated motion is governed by the specified spectrum. A care should be exercised in choosing the observed earthquake motion if the duration can influence the performance of the structure; e.g., the accumulation of inelastic strain either in structural members or in hysteretic vibration control devices depends on the duration of an earthquake motion.

The amplitude of Fourier components is iteratively revised until the response spectral amplitudes of a generated signal becomes close to the target spectral shape using the same procedure used for generation of earthquake motions using random phase angle.

Generation of Site Compatible Ground Motion:
Recent development in engineering seismology makes it possible to generate probable artificial earthquake motions considering the location and historical activity of active faults, the magnitude and propagation of fault fracture, and the propagation of earthquake motions from the fracturing fault to the construction site (Fig. 7). Parameters of an earthquake fault may be length, area, width and inclination angle and direction of a fault, slip length, rise time, and fracture propagation speed.

Three methods are commonly used to generate ground motions for an assumed earthquake source mechanism; (a) the use of semi-empirical Green's functions to simulate the fracture process of a fault and wave propagation from the fault to the construction site based on the earthquake motions observed during smaller and more frequent events originated at similar sources (Irikura [6]), (b) the use of theoretical numerical simulation of fracture process based on assumed geological structure and asperity distribution within the fracturing fault, and (c) the use of empirical relations such as the attenuation of earthquake motions with distance (Kobayashi and Midorikawa [7]). If the geological formation is available through the investigation, the theoretical calculation is useful to estimate long period components of a ground motion; the method, however, tends to underestimate short period components of a ground motion. The semi-empirical method can provide reliable ground motion, but it requires the observed ground motion near the construction site from similar earthquake sources. The empirical relations are normally based on average quantities of ground motion.

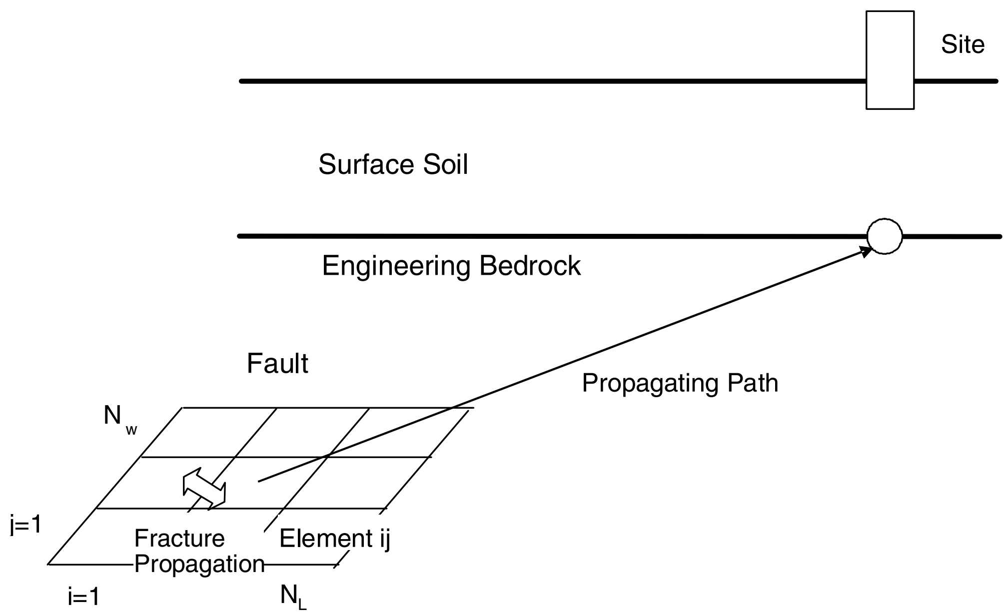


Fig. 7: Generation of Ground Motion from Fault Movements

Amplification of Ground Motion by Surface Geology

The generated earthquake motions at the engineering bedrock are amplified to take into consideration the effect of surface geology above the engineering bedrock (Fig. 8). The change in stiffness and damping of soil with shear strain of soil shall be considered in evaluating the ground motion at the base of the structure. Typical properties of soil are specified in Notification 1457 of year 2000 as shown in Fig. 9. Shear modulus of soil and damping factor are to be modified with the shear strain of each soil layer.

One-dimensional linear response analysis is carried out using an iterative equivalent linearization method revising shear modulus and damping factor with calculated shear strain of each soil layer. Complex transfer function of motions is defined after convergence of iterative linearization. Complex Fourier amplitudes of ground motion at the structure's base are calculated by multiplying complex Fourier amplitudes of ground motion at the engineering bedrock by complex transfer functions of the surface soil layers at each frequency. Time history of a ground motion at the structure's base is constructed from the complex Fourier amplitudes at each frequency.

The dynamic response of soil at the base of a structure may be calculated using nonlinear dynamic analysis of one-dimensional shear beam assuming hysteresis relations (Ramberg-Osgood model) of soil for the generated ground motion at the engineering bedrock.

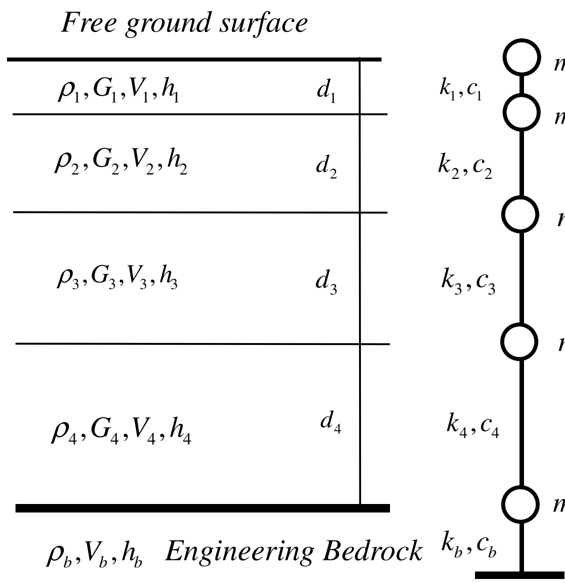


Fig.8: Model for Calculation of Soil Motion

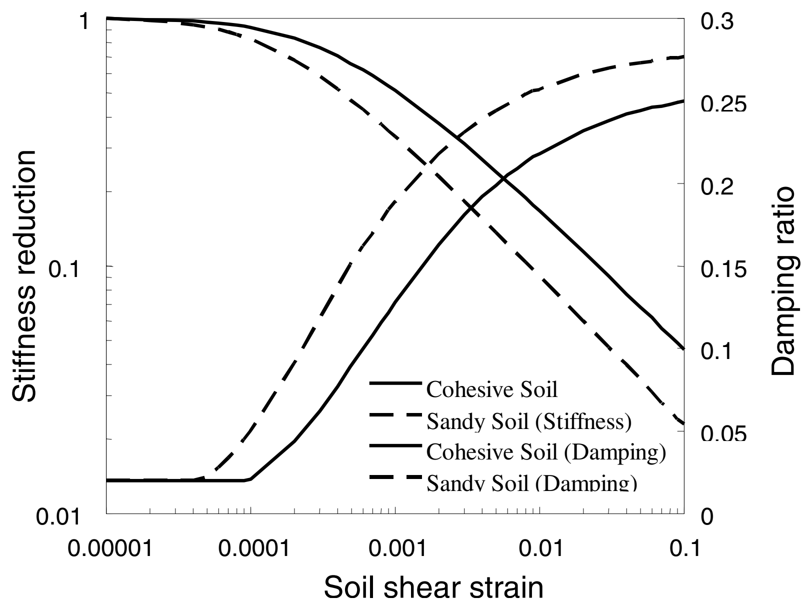
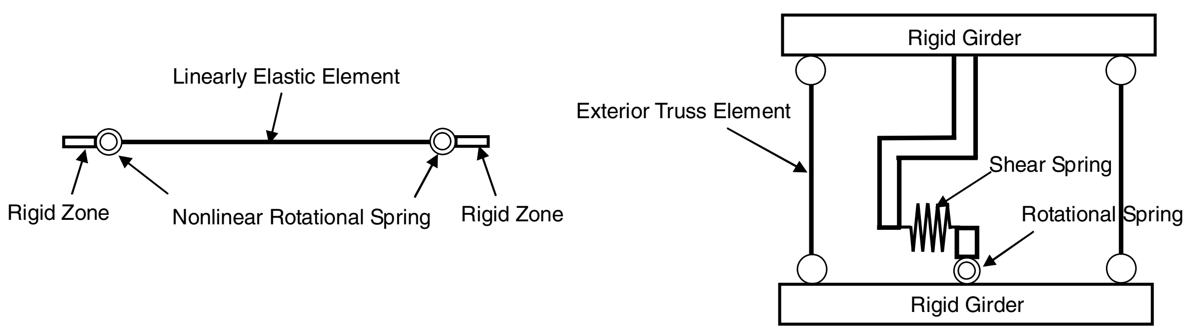


Fig. 9: Stiffness and Damping of Soil with Shear Strain

If the soil layer is not regular in the surface geology, the finite element analysis should be carried out. Furthermore, if the liquefaction is anticipated, special care must be exercised in design piles under earthquake motions.

Nonlinear Static Push-over Analysis
The method of structural modeling for the earthquake response analysis is not specified in the notification. A structure as designed is normally analyzed under monotonically increasing lateral forces considering nonlinear member force-deformation relations. The result is used to formulate a simple mass-spring model for nonlinear earthquake response analysis and to examine the performance of the overall structure under Level-2 earthquake motions.

In the static pushover analysis, girders and columns are idealized by lineal elements with two nonlinear rotational springs at their ends. The rotational spring represents inelastic flexural deformation with stiffness changes at flexural cracking and yielding. The middle lineal element represents elastic flexural and shear deformations of the member (Giberson [8]). Axial deformation with elastic stiffness is considered in the middle elastic element for columns. The effect of varying axial forces on flexural yield resistance of columns is normally considered in the analysis; i.e., flexural yield moment is evaluated for the existing axial force in the member. The beam-column connection is normally assumed to be rigid, but shear deformation may be sometimes considered in the analysis. Structural walls are not commonly used in the super-structure of a high-rise reinforced concrete structure because of the difficulty in the analysis. A structural wall in a story may be represented by two exterior truss elements, rotational and shear springs at the base of the center (Otani et al. [9]). The exterior truss element has low resistance and stiffness in tension representing flexural cracking of the wall panel, and high stiffness and resistance in compression. Figure 10 shows structural models for members.



(a) Member Model for Girders and Columns (b) Member Model for Structural Walls

Fig. 10: Models for Structural Members in Frame Analysis

The moment and rotation relation of member end nonlinear rotational springs may be formulated by assuming the inflection point of the bending moment locates at the center of the linearly elastic element. Cracking and yield moment can be estimated by material properties and geometry at the critical section. Yield rotation calculated by integrating the curvature along the member is normally much smaller than the yield deformation observed in the laboratory. Therefore, an empirical relation is normally used in evaluating the yield rotation (for example, Sugano [10]). The stiffness after yielding is taken as 1/1000 of the elastic stiffness.

Some engineers consider bilinear shear and deformation relation with stiffness change at shear cracking, and also nonlinear axial force-deformation relation ignoring the interaction of axial force and bending resistance.

The distribution of lateral forces in the pushover analysis is normally taken the same as that specified for normal buildings (less than 60 m); higher mode effect is considered in the distribution.

Structural Modeling for Earthquake Response Analysis

The entire three-dimensional structure is sometimes analyzed under Level-2 ground motions using member models and hysteresis models for structural members and actions of all structural members are determined. Even at this age of computers, however, the use of the three-dimensional nonlinear frame analysis for earthquake response calculation is not believed to be desirable by many structural engineers because of the computer time and data examination work.

Lumped Mass-Spring Model

Therefore, a simple mass-spring model is used for the nonlinear response analysis under generated ground motions. Story shear and story drift relation is obtained for each story to construct a multi-mass multi-spring model. Mass is assumed to concentrate at each floor level; i.e., dead load combined with reduced live load for earthquake loading is used to evaluate the floor mass. Story drift is divided into an elastic flexural component and a nonlinear shear component as shown in Fig. 11.

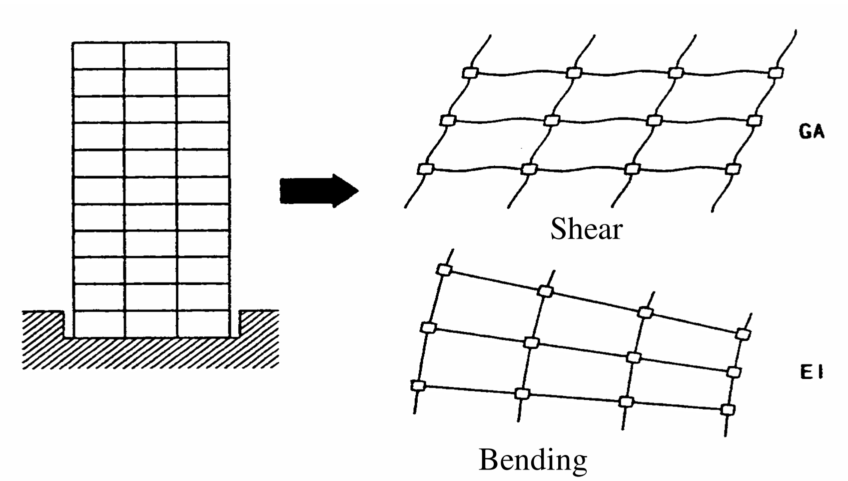


Fig. 11: Bending and Shear Deformation of Frame

The elastic flexural component of story drift is defined by assuming the floor remains plane after lateral deformation; i.e., the vertical deformation of a story is assumed to distribute linearly with the distance from the center of rotation (Fig. 12). The center of rotation is assumed to locate at the geometric centroid of column axial rigidities EA of the story, where E is Young's modulus of concrete and A is cross sectional area of each column. The strain energy of axial forces in all columns in a story is equated with the strain energy of the rotational spring in determining story rotation $\Delta\theta_{ei}$ of floor level i (Fig. 12);

$$\frac{1}{2}\sum_{j=1}^{N} N_{ij}\ \Delta v_{ij} = \frac{1}{2}\sum_{j=1}^{N} EA_{ij}\frac{\ell_{ij}\Delta\theta_{ei}}{h_i}\ \ell_{ij}\Delta\theta_{ei} \tag{9}$$

where, N_{ij} and Δv_{ij} are axial force and axial deformation of j-th column in story i, ℓ_{ij} is the distance from the center of rotation to the j-th column, $\Delta\theta_{ei}$ is equivalent rotation at story i, and EA_{ij} is the axial rigidity of j-th column. It should be noted that the vertical deformation under lateral loading is normally large in the exterior columns and negligible in the interior columns, and that the distribution is not linear in a floor.

After solving the relation for floor rotation, the equivalent flexural rigidity EI_{ei} of a story is evaluated assuming linear moment distribution between two adjacent stories;

$$\frac{EI_{ei}}{h_i} = \frac{\Delta M_i + \Delta M_{i-1}}{2\Delta\theta_{ei}} \tag{10}$$

where, h_i is inter-story height at story i, and ΔM_i is overturning moment at the base of story i.

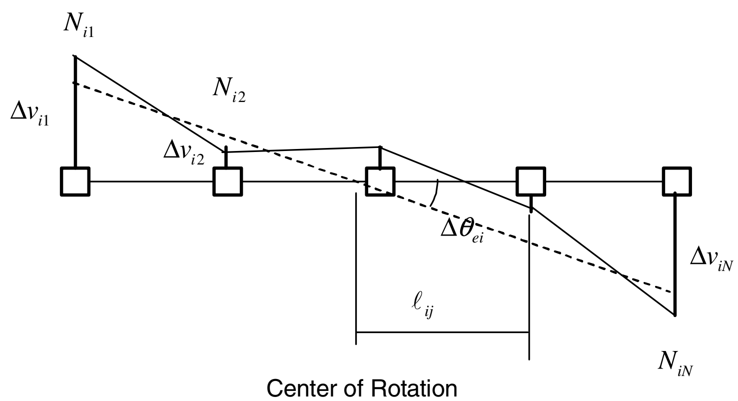


Fig. 12: Distribution of Vertical Floor Displacements and Axial Forces in Columns

The story drift after removing flexural deformation is assigned to the deformation of a story shear spring. The story-shear and shear-spring-deformation relation is idealized by a trilinear relation. The Takeda hysteresis rules (Takeda, Sozen and Nielsen [11]) are used for the story shear spring (Fig. 13).

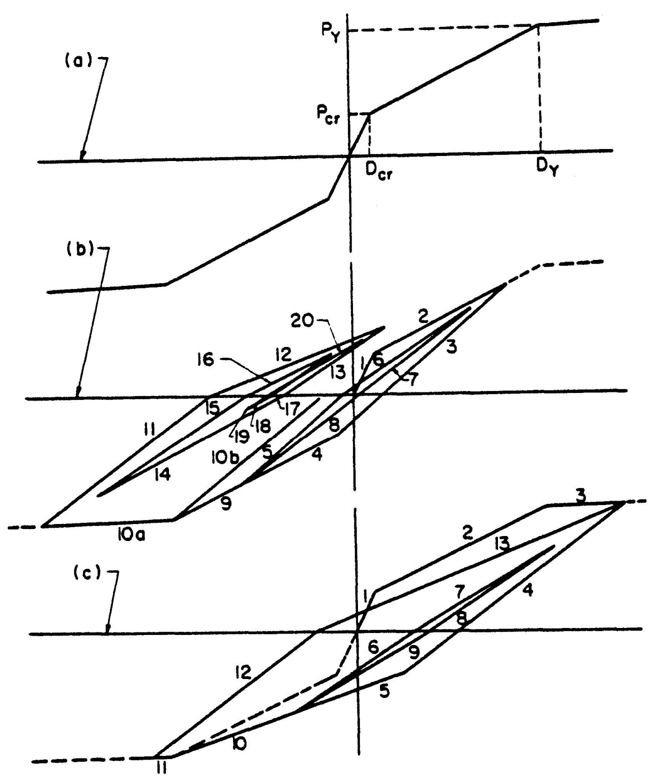


Fig. 13: Takeda Hysteresis Model (Takeda, Sozen and Nielsen [11])

If the structure is irregular in plan, torsional and lateral response should be considered under design earthquake motions. For this purpose, each plane of frames is often represented by a multi-mass-spring model having elastic bending and nonlinear shear stiffness characteristics. These multi-mass-spring models are connected at each floor level by a rigid diaphragm to represent a three-dimensional structure, which is used to examine the torsional effect of an irregular structure.

If the effect of foundation deformation is judged to be important, sway and rocking springs are considered at the base of the structure. The story shear response of a structure normally decreases with the use of soil springs although displacement response may be amplified.

The response of a frame structure may not be represented by the response of a mass-spring model because the story stiffness of the frame structure is closely coupled among stories whereas the story stiffness of the mass-spring is not. This is especially true after yield hinges are formed at the ends of beams. However, in the design of Japanese high-rise construction, the plastic response is carefully controlled to a small value. Therefore, the maximum response of a moment-resisting frame can be closely represented by the response of a mass-spring system. Figure 14 compares the maximum story shear and story drift of a frame and mass-spring models under a Level-2 earthquake motion. The use of a mass-spring model allows the nonlinear earthquake response analysis under various earthquake motions.

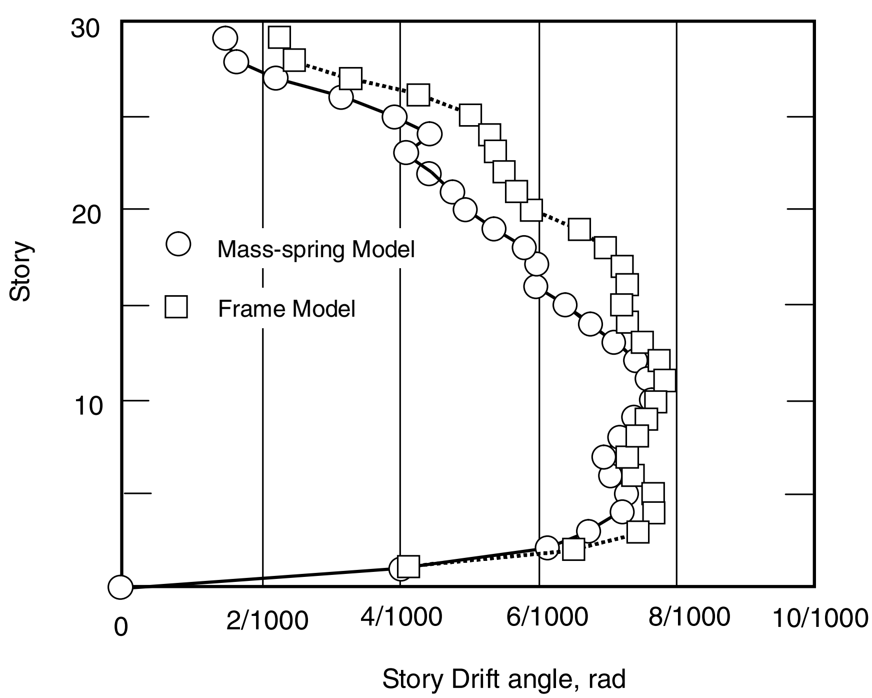


Fig. 14: Comparison of Maximum Response of Mass-spring System and Frame Model (Level 2)

Damping

Damping is normally assumed proportional to instantaneous stiffness of the structure;

$$[c] = \frac{2h_1}{\omega_1}[k^*] \tag{11}$$

where, $[c]$ is damping matrix, $[k^*]$ is instantaneous stiffness matrix, h_1, ω_1 are damping factor and circular frequency of the first mode at the initial elastic stage. The damping factor of the first mode is normally taken as 3 percent for a reinforced concrete building.

Control of Elastic Modulus of Concrete

In a dynamic analysis, the elastic modulus of materials is important because the initial elastic period is proportional to the square root of the elastic modulus of concrete. As a part of the NEW RC research program, Tomosawa et al. [12] proposed an empirical formula to estimate the elastic modulus of concrete as a function of concrete strength, the type of admixture, the type of concrete aggregate and mass density as

$$Ec = k_1 \times k_2 \times 3.35 \times 10^4 \times (\sigma_B / 60)^{1/3} \times (\gamma / 2.4)^2 \qquad \text{(MPa)} \qquad (12)$$

where, k_1 is a factor (=0.95-1.20) representing type of coarse aggregates, k_2 is a factor (=0.95-1.10) representing kind of mineral admixture, σ_B is compressive strength (MPa) of concrete as measured by the standard test of coupon cylinders, and γ is mass density (ton/m3) of concrete. The observed moduli are compared with the estimated values in Fig. 15. A wide scatter of data can be observed. It is important to control the concrete modulus of elasticity during the construction stage if the performance objectives are to be verified by the dynamic analysis.

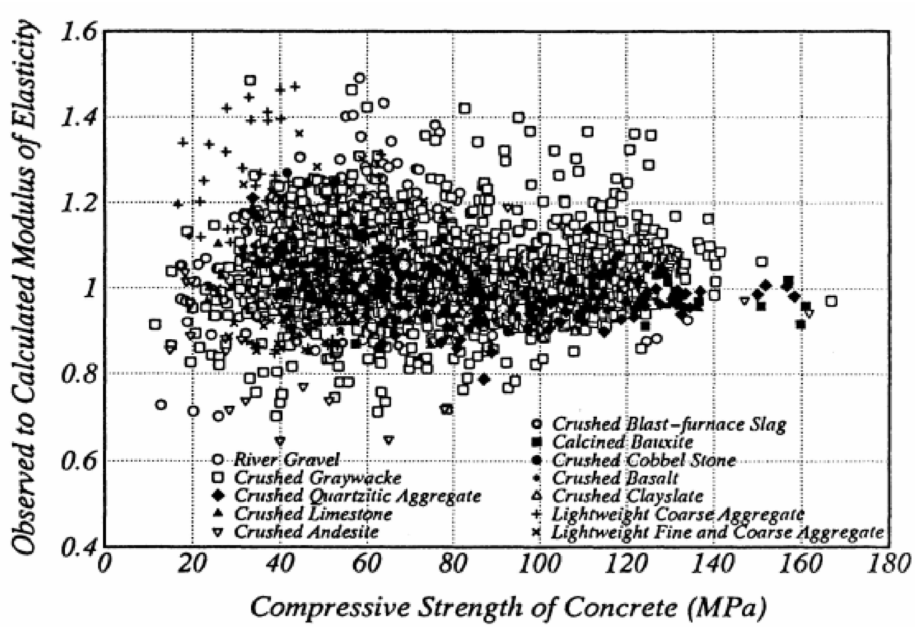


Fig. 15: Observed and Estimated Elastic Moduli of Concrete (Tomosawa et al. [12])

Damage Control of Structure

The performance requirement of a building under Level-1 ground motions is not to cause damage in structural members. The intensity of Level-1 ground motions specified in the notification is by far too small to cause any inelastic stresses in structural members. Therefore, three observed earthquake records normalized to maximum ground velocity of 250 mm/sec are used to examine the performance of the structure for the Level-1 earthquake motions.

The linear response of a multi-mass-spring model is calculated under Level-1 ground motions. Design earthquake story shears are selected, with a prescribed envelope, larger than maximum story shears calculated for all ground motions; the distribution of design story shears is normally taken the same as that of the static pushover analysis. Static linearly elastic analysis of the structure as designed is carried out under the design story shears. Structural members are judged undamaged if the stresses at critical sections are less than the corresponding allowable stresses of materials for the short-term loading. Member dimensions and amount of reinforcement are generally decided by the performance criteria under Level-2 earthquake motions.

The story drift calculated for the mass-spring system must be less than 1/200 so that nonstructural elements should not be damaged by Level-1 earthquake motions.

Structural Safety

The performance requirement of a building under Level- 2 ground motions is not to collapse. For this purpose, the maximum story ductility factor of any nonlinear shear springs of the multi-mass-spring model shall be less than 2.0 and the maximum story drift at any stories less than 1/100. If the story shear is larger than 1/100, the result of special study must be presented indicating that the structural members, non-structural elements and exterior finishing as well as curtain walls are capable of following the deformation without failure.

It is obvious that the overall structural response of the lumped mass-spring system cannot define the collapse state without examining the state of structural members at the maximum response. Therefore, a pushover analysis is used to examine the state of damage in structural members. A displacement at the geometrical centroid (approximately 2/3 of the overall height) of the lateral forces during the static push-over analysis is called a “representative displacement” of the building. Maximum representative displacement of the multi-mass-spring model under the Level-2 ground motions is called as a “response limit displacement (Fig. 16)”.

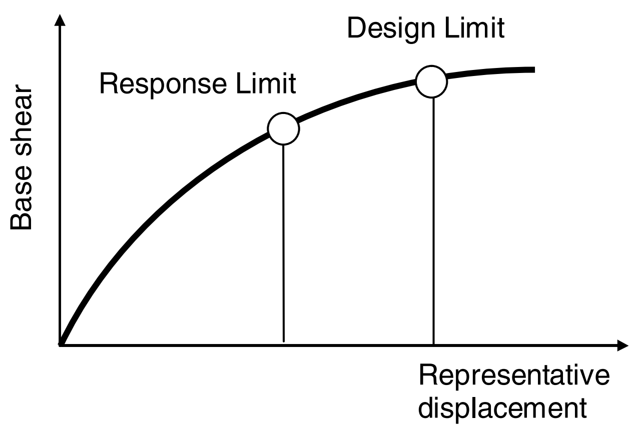


Fig. 16: Response Limit and Design Limit Points

“Design limit displacement” is defined as a point on the representative displacement-base shear curve of the structure under monotonically increasing lateral load such that the area under the curve at the design limit displacement becomes twice as large as that at the response limit point (Fig. 16). The examination of member response at “the design limit point” is necessary because the pushover analysis is carried out under a given distribution of lateral forces which may not represent the distribution of lateral forces during an earthquake excitation and because there exist uncertainty in the definition of ground motion characteristics.

The following response at the design limit displacement is examined: (a) the location of plastic hinge formation, (b) the rotational ductility demand at plastic hinges, (c) the safety margin against plastic hinge formation at column ends, (d) the safety margin against brittle failure in all members, and (e) the maximum axial force level of columns.

The formation of plastic hinges is normally allowed at the ends of girders, at the top of the top-story columns, and at the ends of columns subjected to tensile forces under lateral loading. The rotational ductility factor must be less than 4.0 at girder ends and 2.0 at column ends in tension although experimental evidences have shown that well detailed reinforced concrete member can develop deformation capacity much larger than ductility factor of 4.0 for girders and much larger than 2.0 for columns even subjected to high axial forces.

The column shall be provided with sufficient safety margin against the formation of plastic hinges and brittle failure (shear failure, bond splitting failure and anchorage failure) because they must support the weight of a structure. The compressive axial force shall be limited to a value specified by the structural engineer. Typical safety margin of columns is 1.3 for plastic hinge formation and brittle failure (1.5 for exterior column in compression); the value of safety margin should be selected considering, for example, the safety margin used in strength evaluation against brittle failure, the reliability of structural analysis, the uncertainty in characteristics of earthquake ground motions.

The method to evaluate the resistance of reinforced concrete members against brittle failure is not specified in the notification. “Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Inelastic Displacement Concept” published by Architectural Institute of Japan [13] is normally used to evaluate shear strength of reinforced concrete members, shear resistance at bond splitting failure along the longitudinal reinforcement, resistance at anchorage failure of longitudinal reinforcement.

Additional Studies

The notification requires the examination of following four effects on the earthquake response of a structure; i.e.,

(1) Effect of vertical ground motion,
(2) Effect of orthogonal ground motions,
(3) Effect of phase difference of ground motions, and
(4) Effect of vertical load through horizontal sway.

Effect of Vertical Ground Motion

The vertical motion of an earthquake may excite the vertical oscillation of roofs and slabs, causing additional stresses in the building. This is important in the design of cantilevered members or long-span girders and slabs. The shear of girders caused by vertical vibration must be supported by adjacent columns in the form of axial forces; the axial force in exterior columns sometimes becomes critical when axial forces caused by horizontal and vertical earthquake motions are combined.

Vertical component of artificial ground motions is generated taking into account the response spectrum of vertical ground motions on the engineering bedrock and frequency amplification of vertical motion by the surface geology above the engineering bedrock. The vertical motion has more higher frequency contents than the horizontal motion.

A building structure is idealized by a linearly elastic multi-mass-spring system. A floor mass is assumed to concentrate at the floor level. A spring of a story represents the sum of axial stiffness of columns in the story. The damping matrix is assumed to be proportional to the stiffness matrix, and the first-mode damping factor for vertical vibration is also assumed to be 3 to 5 percent for a reinforced concrete building.

Maximum story axial force of the multi-mass-spring system at each story is distributed to constituent columns proportional to their axial stiffness. Calculated axial forces of columns are combined with those of the pushover analysis at the design limit displacement either by algebraic sum or by square root of sum of squares. Axial forces of columns both in compression and tension are examined.

In some rare occasions, the linearly elastic response analysis of a three-dimensional frame is carried out to find column axial forces under vertical ground motions. In rare cases of long span structures, distributed masses are considered along girders to include the effect of vertical vibration of slabs.

Effect of Orthogonal Ground Motions
An earthquake motion is not limited to the principal direction of a structure. Even if a uni-directional earthquake motion is considered, the motion may not act in the principal direction. When the horizontal ground motion acts in the diagonal direction, axial force level in corner columns may become critical because the corner columns are subjected to axial forces generated by loadings in the two principal directions.

A pushover analysis of frames is first carried out in a direction diagonal to the principal axes in the structural plan. The design limit displacement for diagonal loading is then defined on the base shear-representative displacement curve using the area at the response limit point of the maximum response in one principal direction. In some cases, the maximum response of multi-mass-spring model is calculated in the diagonal direction to define the response limit point.

In some cases, a building is subjected to a level-2 earthquake motion in one principal direction and a level-1 earthquake motion in the other principal direction using a three-dimensional nonlinear frame analysis program.

The axial forces, shear forces and bending moments in columns are examined to satisfy the design performance of the Level-2 earthquake motions.

Effect of Phase Difference of Ground Motions
When a horizontal ground motion propagates at a shear wave velocity with an inclination from the vertical axis, the arrival time of such ground motion varies along the length of structural base. Therefore, the base of a structure is subjected to ground motion of different amplitudes. This is called the phase difference of ground motion (Fig. 17).

The phase difference will excite torsional response of a structure even in the symmetric case (Fig. 18). The effective torsional acceleration may be estimated by the following equation;

$$a_\theta(t) = \frac{\int \rho\, a(t,x)\, x\, dx}{\int \rho\, a(t,x)\, x^2\, dx} \qquad (13)$$

where, ρ is mass per unit length along the length of a structural base, $a(t,x)$ is horizontal acceleration at time t and distance x from the center of mass. The response of a structure can be calculated under the torsional ground motion and combined with the response under horizontal motion.

The effect of phase difference is important when the inclination θ of ground motion from the vertical axis is large and when the length of structural base is long. The inclination angle is normally small when ground motion propagates upward reflecting and refracting at boundaries of soil layers (Snell's Law) because the shear wave velocity of soil is smaller near the surface (Fig. 19). Therefore, the effect of phase difference is believed to be relatively small when the length of base is shorter than 100 m.

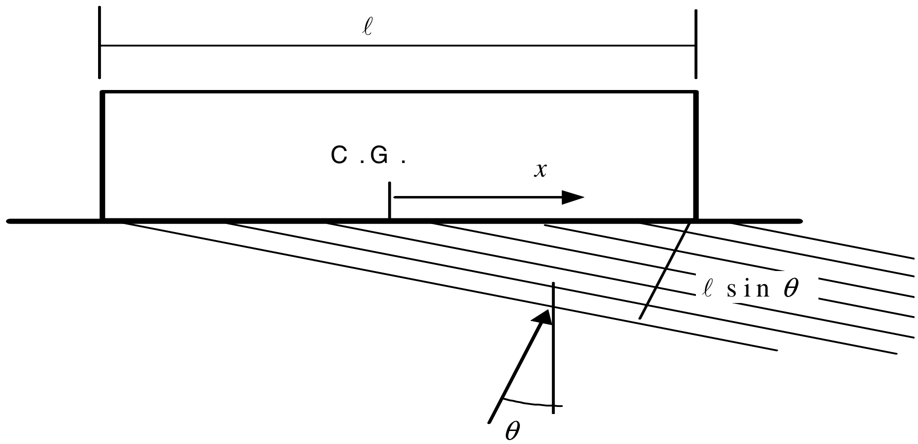


Fig. 17: Phase Difference of Ground Motion

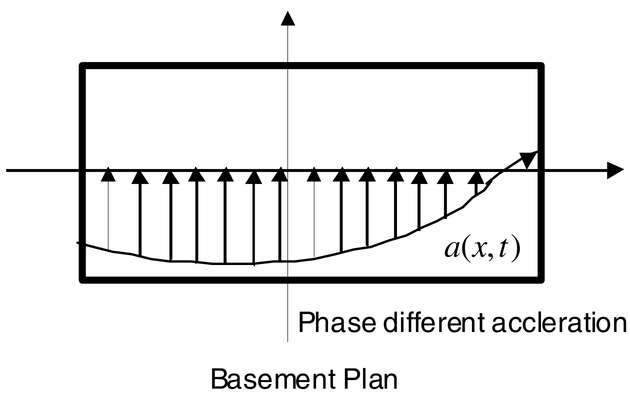


Fig. 18: Horizontal Acceleration Acting on Basement due to Phase Difference Effect

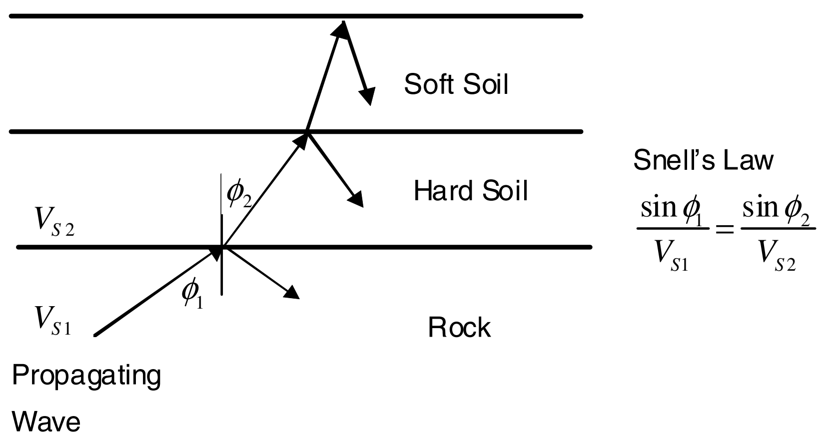


Fig. 19: Reflection and Refraction of Propagating Wave (Snell's Law)

Effect of Vertical Load through Horizontal Sway (P-Δ effect)
The effect of vertical load through horizontal sway is normally ignored in the structural analysis. Therefore, the story drift was examined to remain less than 1/100 even under Level-2 earthquake motion so that the effect could be ignored in the structural analysis.

This effect is also important in evaluating the safety margin against shear failure or flexural yielding in columns in lower stories because the effect increases bending moment and shear in columns subjected to high axial forces especially in external columns where overturning moment by lateral forces causes large variation of axial forces.

DESIGN OF EXTERIOR FINISHINGS AND CURTAIN WALLS

The stresses of external finishing, curtain walls and glass panes caused by out-of-plane wind pressure and inertia forces as well as in-plane inertia forces are examined under Level-2 wind and earthquake forces. The fasteners of curtain walls should be able to resist stresses caused by winds and earthquake motions or should be capable of following the story drift caused by Level-2 earthquake motions.

SUMMARY

Japanese design requirements and the state of practices in design of high-rise reinforced concrete buildings are briefly introduced.

(1) Structural members should not be damaged under permanent gravity loading, which is examined by comparing the member stress and the allowable stress of materials for the long term loading. In addition, deformation and vibration of floor slabs and girders should be examined for serviceability.

(2) Structural members should not be damaged under snow, wind and earthquake events corresponding to a return period of 50 years, which is examined by comparing the member stress and the allowable stress of materials for the short term loading. The design spectrum of earthquake motions is specified at the engineering bedrock, and the amplification of motion by surface geology should be evaluated before the performance evaluation of the structure.

(3) The structure should not collapse under snow, wind and earthquake events corresponding to a return period of 500 years. The action of structural members caused by snow and winds is much smaller than that caused by level 2 earthquake ground motions. Nonlinear dynamic analysis of equivalent multi-mass-spring models is carried out to estimate the maximum structural response under level-2 ground motions. Member performance is examined by the nonlinear static analysis under monotonically increasing lateral forces at a deformation stage much larger than the maximum story drift calculated for multi-mass-spring model. External finishing, curtain walls and glass panes should be safe in level-2 wind and earthquake events.

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