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SEISMIC DESIGN CRITERIA FOR HIGH-RISE R/C BUILDINGS USING HIGH-STRENGTH MATERIALS

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ABSTRACT

Ministry of Construction, Japanese Government, organized the five years national research project from 1988 entitled "The Development of Advanced Reinforced Concrete Buildings using High-strength Concrete and Reinforcement." This paper introduces the seismic design concept developed as a part of the project. The scope is limited to the building height of 200 m, the concrete strength of 60 MPa, and the steel strength of 700 MPa.

KEYWORDS

reinforced concrete; seismic design; high rise building; high strength materials.

INTRODUCTION

Ministry of Construction, Japanese Government, organized a national research project from 1988 to 1993 on "The Development of Advanced Reinforced Concrete Buildings using High-strength Concrete and Reinforcement (The New RC project)". The project involved (a) the development of high-strength materials for reinforced concrete construction, (b) the development of quality control and construction technology suitable for high-strength reinforced concrete structures, (c) the understanding of fundamental material properties, (d) the understanding of member properties under seismic load, and (e) the development of seismic design methodology. The strength of concrete ranged from 36 to 120 MPa, and that of reinforcement from 400 to 1,200 MPa.

As a part of the project, the New RC Structural Design Guidelines (1993) were developed for high-rise reinforced concrete buildings. The Guidelines present design concept and performance criteria of reinforced concrete building structures of regular configuration and of height ranging from 45 to 200m. Because the response characteristics of irregular structures cannot be clearly understood even by using a nonlinear earthquake response analysis of a three-dimensional structure, and reliable strong earthquake motion records containing long period components for the design of structures taller than 200m are not available at present.

The design for gravity load is based on the traditional allowable stress design procedure in Japan. The design against wind load requires the structure to remain elastic (non-yielding) during a strong wind intensity corresponding to a 100 year return period with twice of the standard deviation. However, a seismic design usually governs the structural design of reinforced concrete structures in Japan.

The design against an earthquake motion is, in principle, based on a series of nonlinear earthquake response analyses and a nonlinear static analysis under monotonically increasing lateral forces. The building must satisfy the performance criteria for two intensity levels of earthquake motions; i.e., (a) the building should keep function for an earthquake motion that may occur once in the lifetime of the building, and (b) the building should be repairable and reusable for an earthquake of the largest intensity expected at the construction site. Response drift is also controlled for the two intensity levels of earthquake motions. The locations of yield hinge formation are specified; e.g., the formation of yield hinges in the column is not permitted except at the base of the first–story columns.

REQUIRED PERFORMANCE UNDER EARTHQUAKES

The building must satisfy the required performance for two intensity levels of earthquake motions; i.e., (a) the building should keep function for an earthquake motion that may occur once in the lifetime of the building(Level 1 earthquake motion), and (b) the building should be repairable and reusable for an earthquake of the largest intensity expected at the construction site(Level 2 earthquake motion).
A high–rise building must be designed with a safety margin higher than a low–rise building because the social impact of failure is more pronounced. A large strain at the development of high strength material in a member leads to a structural design governed by drift control rather than by ductility requirements.

Therefore the structural response is controlled by drift (lateral deflection divided by the height). Three limiting drift levels are considered: (1) serviceability limit drift(A), (2) response limit drift(B), and (3) design limit drift(C). The serviceability limit drift is used to control structural and non–structural damage under Level 1 earthquake motion; the response limit drift is intended to control the deformation under the possible strongest intensity earthquake (Level 2); and the design limit drift is used to examine the deformation (ductility demand) at yield hinge regions and to determine the design force level (resistance demand) in non–yield hinge regions under the probable maximum response deformation. The serviceability and response limit drifts may be selected based upon the building use by a structural engineer, but should not exceed 1/200 and 1/120, respectively. The design limit drift is defined as a structural drift at which the area under a base shear–structural drift curve of a structure becomes two times that at the response limit drift.

The required performance of a structures is illustrated in Fig.1.

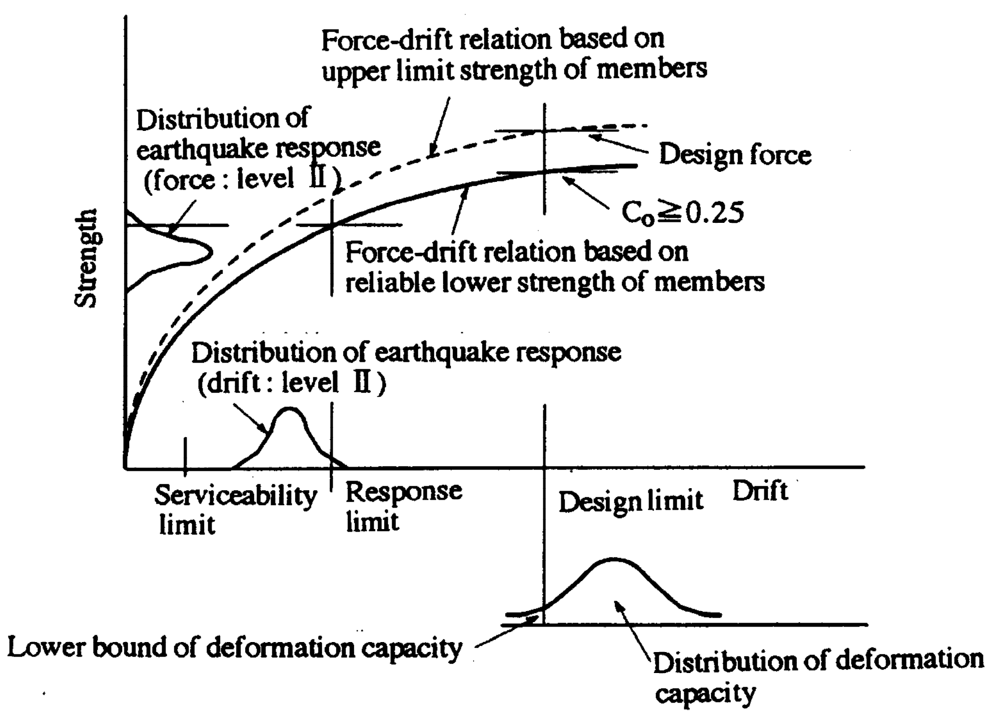


Fig. 1. Required performance of structures

DESIGN EARTHQUAKE MOTIONS AND FORCES

Design Earthquake Motion

Characteristics of a design earthquake motion are first evaluated at the seismological bedrock (shear wave velocity of higher than 3,000 m/sec), and then at the engineering bedrock (shear wave velocity of 400 m/sec) which supports the foundation of a building. The intensity and duration of an earthquake motion at the seismological bedrock should be determined, in principle, taking into account (1) expected magnitude of earthquakes, (2) epicentral distance, (3) probability of earthquake occurrence, and (4) lifetime of the building. A proper amplification of wave by rock and soil layers above the seismological bedrock should be considered in evaluating an earthquake motion at the engineering bedrock. Characteristics of input earthquake motions at the structural base should be carefully evaluated taking into consideration of (1) surface geology at the site and (2) type of foundation of the structure.
For the life safety under an earthquake disaster and the preservation of social properties, the minimum earthquake intensity must be specified. Two levels of intensity are used for design earthquake motions

Level 1 Earthquake Motion

Level 1 earthquake motion is expected to occur once during the lifetime of a building (earthquake motion of an expected return period of approximately 100 years),

Level 2 Earthquake Motion

Level 2 earthquake motion is the largest earthquake motion that may occur at a construction site (earthquake motion of an expected return period of approximately 400 years).
For an assumed building lifetime of 100 years, the probability of earthquake intensity exceeding the design level is 60 percent and 20 percent for Level 1 and 2 earthquake motions, respectively.

Level 2 earthquake motions in Tokyo area were simulated utilizing various prediction techniques for a magnitude 8 class earthquake and a response spectrum of ground motion on the exposed (free surface) engineering bedrock was simulated (Fig.2). The intensity was found comparable to or slightly stronger than the earthquake intensity commonly used in the design of high-rise buildings in Japan. Design earthquake motions must be artificially simulated to satisfy the specified response spectrum with large amplitudes in a long period range. In addition, several observed earthquake motions with an appropriate frequency content must be used in design.

Required Static Earthquake Force for The Design Limit Drift

The required lateral force resistance in terms of a base shear coefficient C_B of a structure at the design limit drift is specified below:

$$C_B = Z\ Rt\ Co \tag{1}$$

in which, Z: seismic zoning factor, Rt: factor representing dynamic characteristics of a structure, dependent on fundamental periods of the structure and the soil, Co: standard value (=0.25) of base shear coefficient.

If seismic activity at a construction site is not investigated, the seismic zoning factor Z should be unity. The vibration characteristic factor Rt is extrapolated from the expression defined by the Building Standard Law Enforcement Order (Eq. 3) for three types of soil(Fig.3).

$$\begin{aligned} &Rt=1.0 && \text{for } T< Tc \\ &Rt=1.0-0.2((T/Tc)-1)2 && \text{for } Tc< T<2Tc \\ &Rt=1.6(Tc/T) && \text{for } 2Tc<T \end{aligned} \tag{2}$$

where T: fundamental period of a structure, Tc: critical period of subsoil (0.4 sec for rock, stiff sand and gravel; 0.6 sec for others; and 0.8 sec for alluvium). The fundamental period of a structure may be estimated on the basis of the initial elastic stiffness.

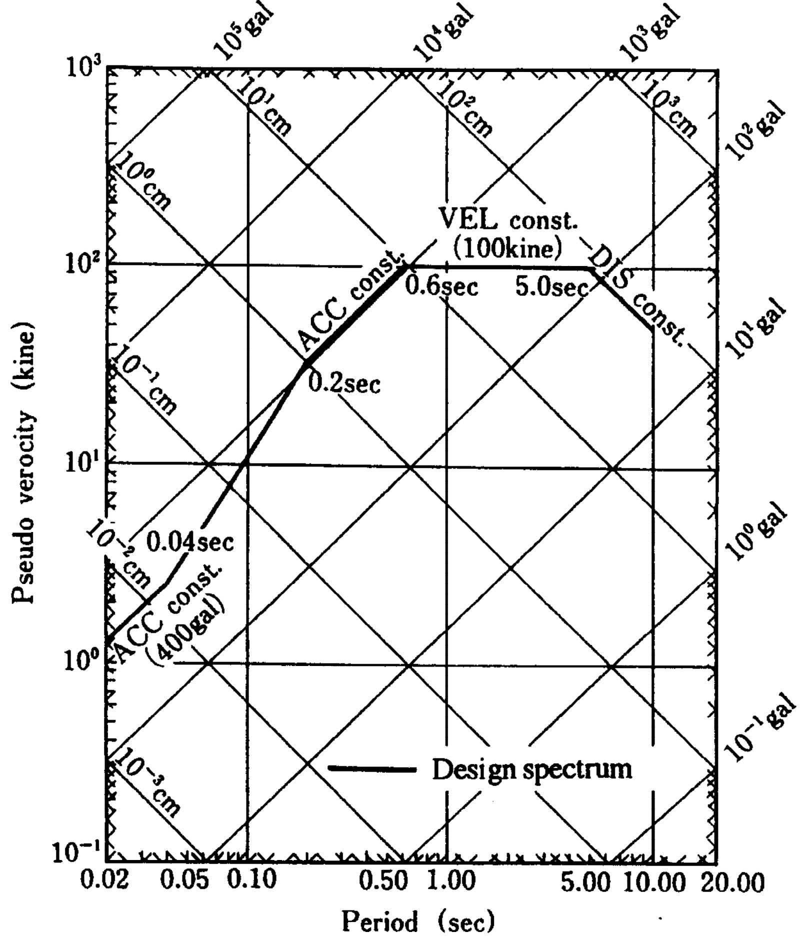


Fig. 2. Design response spectrum on exposed engineering foundation

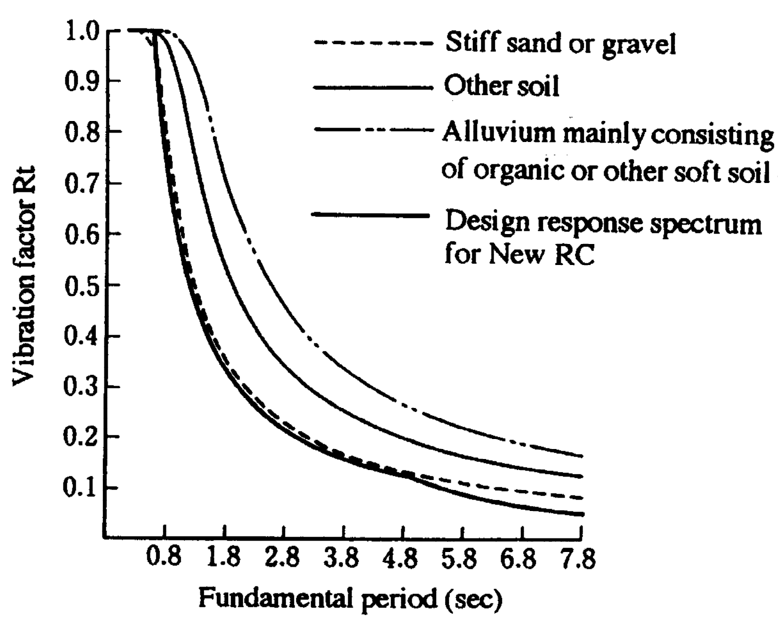


Fig. 3. Rt curve defined in building standard low of Japan and design response spectrum of earthquake motion for New RC

The distribution of static lateral forces must simulate the story shear distribution expected during an earthquake. The response of a high–rise building is influenced by the contribution of higher modes of oscillation, which is influenced by an assumed type of damping. It is not desirable to assumed large damping factors for the second and third modes.

Direction of Seismic Design

The safety of a building should be examined for horizontal earthquake motions occurring in all possible directions. The principal directions must be determined for a bi–directional horizontal earthquake motion. The motion only in the principal direction may be assumed to act on a structure in any horizontal direction; the minor principal earthquake motion may be ignored. Some additional design forces are shown to be sufficient to cover the orthogonal effect.

PERFORMANCE CRITERIA UNDER EARTHQUAKE CONDITION

Performance Criteria

A structure must satisfy serviceability performance criteria for Level 1 earthquake motions and safety performance criteria for Level 2 earthquake motions.

Criteria for Level 1 Earthquake Motion

The serviceability criteria are that (1) story drift (inter–story deflection divided by the inter–story height) should be less than the serviceability limit drift, (2) any structural member should not, in principle, develop yielding and (3) non–structural elements should not be damaged. A boundary girder, connected to a structural wall, or a short girder may develop yielding; however, the yielding should not deteriorate the earthquake resistance of the structure. A non–structural element would not be damaged if a structural drift is limited within the serviceability limit drift. Cracks in a structural member might be easily repaired by injection of epoxy resin. Therefore, the structure is expected to be usable immediately after the earthquake.

Design Criteria for Level 2 Earthquake Motion

The safety criteria are that (1) structural drift (lateral deflection at the two–third height of a building divided by the height at the level) should be less than the response limit drift, (2) maximum story drift should be less than 1.5 time the response limit drift to avoid the concentration of deformation in a limited number of stories, (3) the location where yielding is permitted must maintain its full resistance, (4) the location where yielding is not permitted should not develop yielding, and (5) brittle failure, such as shear failure or bond splitting failure along the longitudinal reinforcement, should not take place in any member. The damage in structural elements is expected to be repairable, and the structure may be used after the repair work.

Flexural yield hinges are permitted to form (1) at the ends of girders and (2) at the top of top–story columns, where large ductility and stable hysteresis energy dissipation should be expected. Columns and structural walls, in general, must not develop yielding because they are difficult to develop large ductility due to the existence of high axial load in a column or due to wide geometry of a wall; furthermore, the failure of a column or a structural wall may lead to the collapse of the structure. However, yield hinges may form at the base of first–story columns and structural walls to allow the formation of an overall structural mechanism under lateral forces. An exterior column subjected to net tension force may be allowed to develop yielding because the bending moment resistance is small and because a large ductility can be expected under tensile axial force; however the column should not develop yielding under compressive axial force when the direction of lateral forces are reversed.

Design Criteria at The Design Limit Drift

The performance of a structure is also specified for the response calculated by a nonlinear static analysis under monotonically increasing lateral forces to compensate the uncertainty in the characteristics of earthquake motions, the reliability of analytical methods, and a limited number of earthquake motions used in the response analysis. Up to the design limit drift, (1) the location where yielding is permitted must main–

tain its full resistance, (2) the location where the yielding is not permitted should not develop a yielding hinge, (3)brittle failure should not take place in any member, and (4) the lateral resistance expressed in terms of a base shear coefficient (first story shear divided by the total structural weight) should be larger than the static earthquake force(Co=0.25). The base shear coefficients of New RC trial design building(denoted as open square) and existing high-rise building in Japan are plotted in Fig. 4. Standard base shear coefficient (Co) is found to be reasonable.

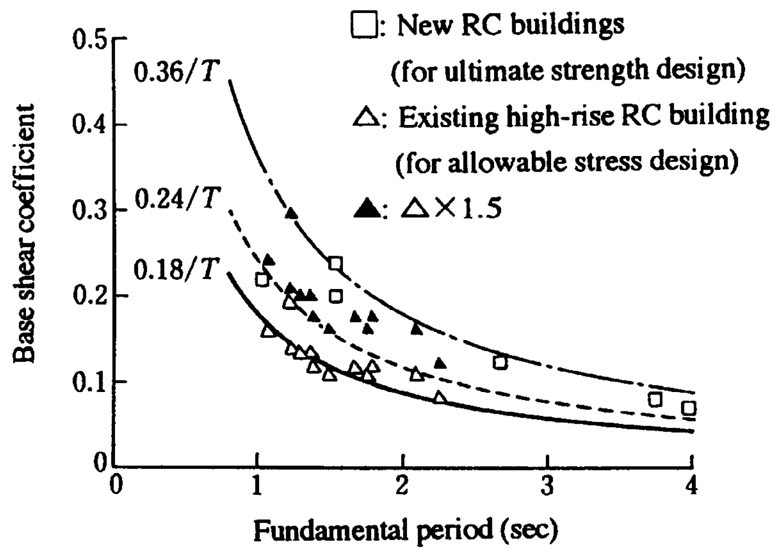


Fig. 4. Base shear coefficient of New RC and existing high-rise RC buildings

DEPENDABLE AND UPPER BOUND STRENGTH OF STRUCTURAL MEMBER

The reliable and the upper bound resistance of members are determined to confirm a probability of non-exceedance of 0.90 on a statistical basis of experimental data. If ratios of the observed to the calculated resistance are assumed to distribute in a normal distribution, there liable and the upper bound resistance is estimated as follows from the calculated resistance R, average ratio AR of the observed to the calculated resistance and coefficient of variation COV of the ratios;

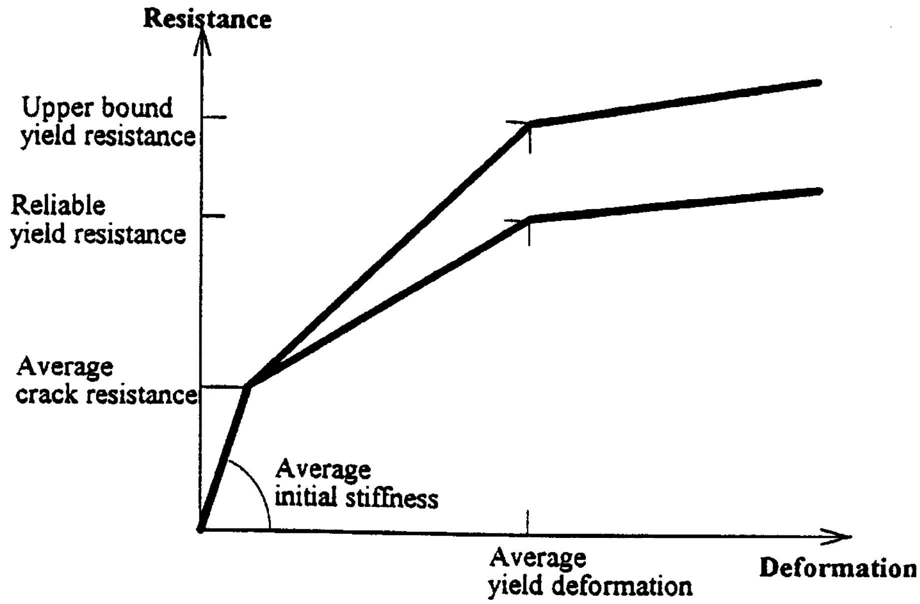
$$\text{Reliable resistance} = \text{R} \times \text{AR} \times (1.0 - 1.28 \times \text{COV}) \tag{3}$$

$$\text{Upper bound resistance} = \text{R} \times \text{AR} \times (1.0 + 1.28 \times \text{COV}) \tag{4}$$

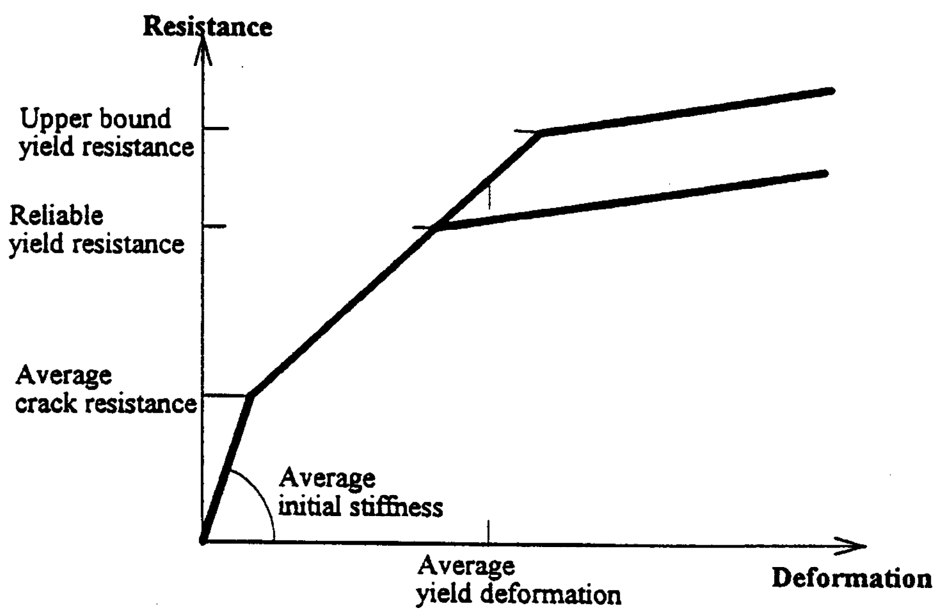
The reliable resistance must be used in all members when response drift of a structure is examined in the earthquake response analysis under Level 1 and 2 earthquake motions and when lateral force resisting capacity of a structure available at the design limit drift is examined in the static analysis under monotonically increasing lateral forces. The upper bound resistance is used at the location of allowed yield hinges in the static analysis when design actions are determined for a region other than allowed yield hinge region or when the brittle failure of a member is examined.

The stiffness of a reinforced concrete member may be assumed to change at cracking and yielding. A yield point in the guidelines is defined as the point at which the stiffness degrades significantly under monotonically increasing force; however, the yield resistance may be estimated by a routine procedure for the tensile yielding of longitudinal reinforcement.

Average values for the initial stiffness , cracking moment and yielding deformation are used whereas the reliable (lower bound) and upper bound yield resistance are used to take into account the variation of material strength and the reliability of evaluation methods. The stiffness characteristics of a girder and a column are assumed as shown in Fig. 5. For the experimental data studied, the yield deflection for columns did not change appreciably with yield resistance for a wide variety of parameters, while the yield deflection of girders increased with yield resistance.



(a) Column



(b) Girder
Fig. 5. Stiffness characteristic model

SUMMARY

The Structural Design Guidelines developed as a part of the New RC project in Japan were briefly outlined in this paper. The Structural Design Guidelines present design performance criteria for a reinforced concrete building structure of regular configuration and of height ranging from 45 to 200m, using concrete of strength up to 60 MPa and longitudinal reinforcing steel of strength up to 700 MPa. The design against earthquake motions is, in principle, based on a series of nonlinear earthquake response analysis and a nonlinear static analysis. The building must be designed for two intensity levels of earthquake motions; i.e.,(a) the building should be serviceable for an earthquake motion that may occur once in the lifetime of the building, and (b) the building should be reusable for an earthquake of the largest intensity expected at the construction site. Due to high seismicity in the region and due to the use of high strength materials, drift limits govern the design rather than ductility requirements. The allowable locations of yield hinge formation are specified to improve the performance.

ACKNOWLEDGMENT

The Structural Design Guidelines for the New RC building were developed by Design Guidelines Committee (Chairman: Dr.Tsuneo Okada) under the Research Coordinating Committee(Chairman: Dr. Hiroyuki Aoyama, Professor Emeritus of the University of Tokyo). The Guidelines were drafted and discussed by a sub-committee chaired by Dr. Tetsuo Kubo of the Nagoya Institute of Technology. The authors want to express their sincere gratitude to those who were involved in the development of the guidelines.

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