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PRACTICA DE DISEÑO Y
CONSTRUCCION EN EL JAPON

JAPANESE PRESS DESIGN GUIDELINES FOR REINFORCED CONCRETE BUILDINGS

Shunsuke Otani

E PREVENCION DE DESASTRES
MEXICO

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CUADERNOS DE INVESTIGACION

Práctica de Diseño y Construcción en el Japón

P R E S E N T A C I O N

Uno de los objetivos del Convenio de Cooperación Técnica entre la Agencia de Cooperación Internacional del Japón (JICA) y el Centro Nacional de Prevención de Desastres es la difusión en México de tecnologías y metodologías de diseño y construcción de estructuras en el Japón.

Estas prácticas y experiencias son descritas en conferencias o seminarios dictados, o bien mediante la traducción al castellano de los textos originales del japonés, por los expertos japoneses de corto y largo plazo que colaboran en las actividades de investigación del CENAPRED.

Para lograr una difusión más amplia de las tecnologías y metodologías del Japón, el CENAPRED ha emprendido la publicación de esta serie como parte de los Cuadernos de Investigación.

CONTENIDO

PROLOGO

Varias son las filosofías que han surgido recientemente para el diseño sísmico: diseño por capacidad, por desplazamiento, o con base en consideraciones energéticas. De los métodos anteriores, el más desarrollado hasta la fecha es el adoptado por Nueva Zelanda e incorporado en las guías de diseño presentadas, de manera simple y completa en este trabajo, por S. Otani: el diseño por capacidad. Según este enfoque, el diseñador debe seleccionar un mecanismo estable con articulaciones plásticas en elementos, y debe diseñar todos ellos (independientemente del nivel de deformación esperado) de modo que no exhiban fallas de tipo frágil (por cortante y adherencia).

Aunque desarrolladas dentro de un programa de investigación en estructuras precoladas de concreto, las guías son aplicables para estructuras monolíticas de concreto reforzado. Para el Japón, estas guías representan un avance notable en filosofías de diseño, al adoptar un criterio por estados límite. En el diseño por sismo se señalan dos niveles de intensidades: un sismo de servicio que puede ocurrir varias veces en la vida útil de la estructura, y uno de falla que se espera a lo más una vez. Las guías son aplicables para materiales, concreto y acero de refuerzo, con resistencias normales.

Lo anterior no constituye algo sustancialmente distinto de lo que es considerado en otros países, incluyendo a México. Sin embargo, las guías señalan que para determinar el desplazamiento de entrepiso para cada nivel de diseño (o intensidad) se empleen análisis en los cuales se considere el intervalo de comportamiento no lineal de los materiales; esto se aparta de lo convencional. Consistente con el avance y confiabilidad de los análisis sísmicos no lineales, las guías se limitan a estructuras regulares y sin torsión, poco esbeltas, y desplantadas en suelos con compresibilidades baja y media. Un inconveniente de la metodología planteada es que la mayoría de las firmas de diseño de ingeniería no están preparadas para ejecutar análisis no lineales que son complejos y consumen mucho tiempo.

Como deformación máxima se acepta 0.010, valor que se reconoce sin fundamento teórico. Un aspecto interesante es la mención del nivel de resistencia que deben mantener los elementos resistentes a carga vertical cuando el sistema resistente a cargas laterales alcance la deformación máxima permitida ("proof deformation"). El colapso de varias estructuras en el sismo de Northridge 1994 (Los Angeles, EUA) puso en evidencia la necesidad de considerar la compatibilidad de desplazamientos laterales.

Otra aportación interesante del trabajo expuesto por Otani es sobre los efectos de los movimientos bidireccionales en la respuesta de estructuras. El colapso del edificio del Condado del Valle Imperial en EUA atrajo la atención de los ingenieros estructuristas sobre la vulnerabilidad de columnas ante desplazamientos laterales alternados aplicados biaxialmente. Un creciente número de ensayes de laboratorio con leyes de cargas biaxiales se ha efectuado para entender el comportamiento de columnas, de nudos viga–columna y de otros componentes de estructuras. Las nuevas guías de diseño japonesas establecen las especificaciones que las columnas deben cumplir bajo desplazamientos uniaxiales y biaxiales, en función de las deformaciones plásticas del mecanismo de fluencia.

Aunque varios de los requisitos de refuerzo (como separaciones de estribos, por ejemplo) carecen de fundamento riguroso, las aportaciones y las filosofías de diseño incorporadas hacen del texto de Otani un documento valioso y de consulta periódica.

Japanese PRESSS Design Guidelines for Reinforced Concrete Buildings

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Synopsis: The paper briefly introduces an ultimate strength design method for reinforced concrete buildings on the basis of the capacity design concept. A design guidelines was developed in Japan as a part of the U.S.–Japan PRESSS (Precast Seismic Structural System) project. The design for earthquake loading is based on ultimate strength design procedure for serviceability limit state and ultimate limit state. This paper introduces the concept of earthquake resistant design for the ultimate limit state using a nonlinear static analysis under monotonically increasing load.

INTRODUCTION

The first Japanese building code, Urban Building Law, was promulgated in 1919, to regulate building construction in six major cities at the time. Structural design, based on the allowable stress design procedure, was outlined in the Urban Building Law Enforcement Regulations enacted in 1920. Earthquake resistant design with a seismic coefficient of 0.10 was introduced in 1924 in the Urban Building Law Enforcement Regulations after the 1923 Kanto Earthquake. The Urban Building Law was enforced gradually in an increased number of cities.

The Building Standard Law (Building Center of Japan, 1990) was proclaimed in 1950 to regulate all building construction throughout Japan. Structural design method was provided in the Building Standard Law Enforcement Order, in which two levels of allowable stresses were introduced for (a) long–term normal gravity loading conditions and (b) short–term unusual (earthquake and wind) loading situations. The allowable stresses for the short term loading were increased to full yield strength for reinforcement and to two–third compressive strength for concrete; i.e., allowable flexural resistance of beams under short–term loadings became comparable to the ultimate flexural strength. The design seismic coefficient was increased to 0.20 reflecting the increase in the allowable stresses.

The Building Standard Law Enforcement Order, revised in 1981 (Building Center of Japan, 1990), adopted a two–level design procedure; i.e., (a) traditional allowable stress design and (b) examination of ultimate lateral load resistance of each story.

The Architectural Institute of Japan (AIJ, 1933) published "Standard for Structural Calculation of Reinforced Concrete Structures" in 1933, based on allowable stress design procedure. After many efforts to develop an ultimate strength design procedure over the last two decades, AIJ (AIJ, 1990) published "Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings based on Ultimate Strength Concept" in November, 1990. The AIJ Guidelines has not been approved for general use by the Ministry of Construction.

The basic philosophy outlined in the AIJ Guidelines was to avoid negative performance of a building during an intense earthquake, such as,

(a) large plastic deformations,

(b) concentration of damage in limited locations, and

(c) brittle failure.

Therefore, the AIJ Guidelines proposes to design a building to develop a specified total yield mechanism under a design earthquake. The locations of yield hinges are selected to ensure sufficient structural deformation capacity; the strength of yield hinges is determined to develop a lateral resistance required to limit the overall deformation of the building. The locations, where yield hinges are not intended, are provided with sufficient resistance against brittle failure. The concept is similar to the one used in the capacity design, developed in New Zealand (New Zealand Standards Association, 1982).

The concept of the AIJ Guidelines was further extended in the current PRESSS design guidelines, which introduces serviceability and ultimate limit state design requirements. Two separate procedures are prepared for earthquake resistant design; i.e.,

(a) a procedure using static nonlinear analysis of a building under monotonically increasing lateral loads, and

(b) a procedure using a linearly elastic analysis and a simple limit analysis for a building less than 31 m in height.

This paper introduces the earthquake resistant design procedure using an incremental static nonlinear analysis. Some values of design coefficients are tentative, and may be revised with development.

It should be noted that the Ministry f Construction has not authorized the use of this guidelines in practice.

SCOPE OF PRESSS GUIDELINES

The PRESSS guidelines is intended to provide minimum requirements for the structural design of cast-in-situ reinforced concrete (RC) buildings with/without precast reinforced concrete (PCa) structural elements. PCa elements and their connections must be properly designed and detailed to exhibit behavior as good as or superior to that of corresponding monolithic reinforced concrete assemblages. For this purpose,

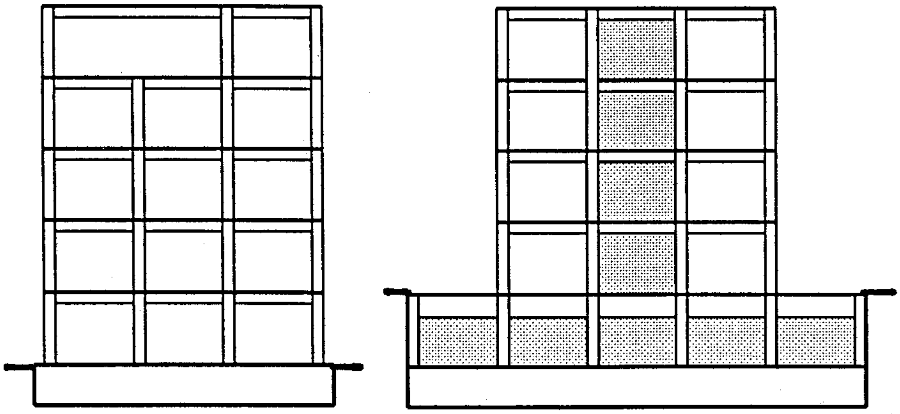
(a) Standard for the Evaluation of PCa Connection Performance,

(b) Design Manual for PCa Connections, and

(c) Guidelines for Construction and Quality Control of PCa Construction,

were developed along with the Design Guidelines.

The application is limited to ductile moment resisting frame buildings of regular structural configuration, with/without continuous structural walls. A girder must be continuous from one end to the other end of a building; a column must be continued from the foundation to the top-story of the column; a structural wall must be continuous from the foundation to the roof level. An example of buildings within the scope is shown in Fig. 1.



(a) Frame Building (b) Frame Building with Structural Wall
Fig. 1: Example Buildings

Building Height and Shape -- The height of a building is limited to 60 m because the Building Standard Law (Building Center of Japan, 1990) requires a special approval of the design of a building taller than 60 m by the Minister of Construction. The structure must be regular in plan and along the height. The aspect ratio (structural height to width ratio) is limited to 4.0; if an aspect ratio exceeds the limit, exterior columns must be carefully designed against a large variation of axial load due to overturning moment.

Structural Regularity -- The structural regularity is judged, in conformance with the Building Standard Law Enforcement Order (Building Center of Japan, 1990), by a stiffness ratio R_{si} and an eccentricity ratio R_{ei} of story i, defined by Eqs. (1) and (2);

$$R_{si} = r_{si} / \underline{rs} \tag{1}$$

$$R_{ei} = e_i / r_{ei} \tag{2}$$

where r_{si}: reciprocal of inter-story drift angle at story i, calculated by linearly elastic analysis under design earthquake loading for serviceability limit state, $\underline{rs}$: arithmetic mean of all r_{si}'s, e_i: distance of eccentricity from the center of stiffness to the center of vertical gravity loads at story i, and r_{ei}: elastic radius of gyration at story i, defined as the square root of the second moment of column and wall stiffnesses about the center of rigidity divided by the sum of column and wall stiffnesses. The stiffness ratio should be equal to or greater than 6/10 and the eccentricity ratio equal to or less than 15/100 to be qualified as a regular structure.

Soil -- A structure must be supported on a firm or inter-mediate soil.

Materials -- Design concrete strength is limited to 21 to 36 MPa for normal concrete, and 21 to 27 MPa for light-weight aggregate concrete. The grades of deformed bars are SD295A, SD295B, SD345 and SD390, in which the number indicates specified yield stress in MPa. The bar sizes are limited to 10 to 41 mm in nominal diameter.

High-strength prestressing bars (yield stress ranging from 700 to 1,300 MPa) may be used as shear reinforcement if the use of specific products with their design specifications is approved by the Minister of Construction.

PERFORMANCE CRITERIA

The performance criteria of super-structure, basement and foundation structure of a building, as designed, are specified for serviceability and ultimate limit states.

Serviceability Limit State -- A structure must be serviceable without major repair work immediately after a medium intensity earthquake motion, which may be expected to occur several times during the use of the building. A structural member must not yield nor non-structural element and mechanical facilities should be damaged. A story drift must be limited to 1/200 of the story height under the design earthquake load.

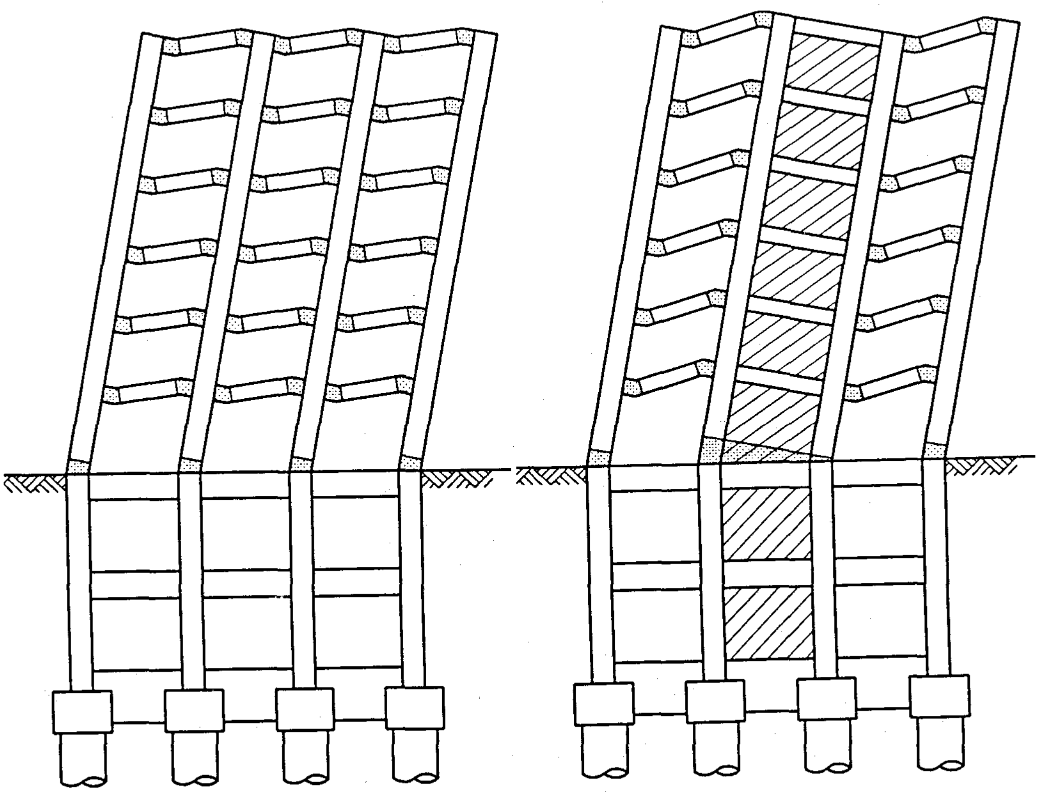
Ultimate Limit State -- A structure must be usable, even with an extensive repair work, after a high intensity earthquake motion, which may occur once in the lifetime of the structure.

A moment-resisting frame structure with continuous structural walls is allowed to develop flexural yielding (see Fig. 2) at (a) the ends of beams at the second floor and above, (b) the base of first-story columns, (c) the base of structural walls, (d) the top of the top story column, (e) the top and bottom of an exterior column under tensile force developed by the earthquake loading, (f) the end of a foundation girder directly connected to a structural wall which is supported by direct foundation, and (g) the end of first-floor girders not connected to an exterior column. A structural wall supported by direct foundation may be up-lifted on the tension side by the lateral loading; however, the uplifting of a structural wall supported by a pile foundation is not allowed to up-lift because the ultimate pull-out resistance of the foundation cannot be accurately evaluated by the state-of-the art.

It has been known that a structure oscillates dominantly in the fundamental mode during a strong earthquake. Therefore, the structure can be designed to form a specified yield mechanism under the fundamental mode oscillation. The yield mechanism must be of total collapse mechanism type (Fig. 2), which develops plastic hinges throughout a structure and develops a uniform inter-story drift over the entire structural height. A partial yield mechanism (Fig. 3), on the contrary, develops plastic hinges in a limited number of stories. Nonlinear earthquake response analyses indicated that the overall deflection of a structure is comparable for different distributions of damage within a building. Therefore, the amount of plastic deformation at a yield hinge is smallest in a total yield mechanism.

Yielding hinges are also desired at beam ends because (a) it is easy to develop a large plastic deformation, (b) a large and stable hysteresis energy can be dissipated, and (c) the failure of a beam will not lead to the collapse of the structure. Furthermore, simultaneous yielding at all beam ends can dissipate substantial hysteretic energy as a structure. On the other hands, the yielding at column ends may lead to a partial yield mechanism (Fig. 3), and a significant damage in a column may lead to the collapse; columns are difficult to develop a large plastic deformation. However, an exterior column subjected to tension and a top story column subjected to low axial load can develop a large ductility, and may be allowed to yield in an earthquake.

A structural wall must be planned to yield in flexure or by uplifting at the base. Both yield mechanisms are believed to be ductile. With structural walls, the deformation of a structure is controlled by the structural walls and tends to distribute almost uniformly along the structural height, hence the yielding in columns may be permitted in a frame–wall structure. However, columns are not encouraged to yield because they normally carry high axial load and their deformation capacity is limited.



Frame Building (b) Frame Building with Wall

Fig. 2: Possible Yield Mechanisms of Structures

The structure must develop lateral resistance greater than a specified value at the design limit deformation (Limit Deformation). The region, where yielding is expected, must maintain the resisting capacity to the design proof deformation (Proof Deformation). Shear and bond failure must be prevented in any member to the Proof Deformation.

The Limit Deformation is the maximum deformation expected under a high intensity design earthquake motion, while the Proof Deformation (two times the design limit deformation) is the upper bound deformation taking into consideration the uncertainty in characteristics and intensity of design earthquake motions, and the reliability of structural analysis and member strength evaluation.

The foundation and basement must transfer the actions by gravity and lateral earthquake loads from the super–structure to the ground. Yielding, as a general rule, must not develop in the foundation structure, including foundation girders, foundation slabs, and piles.

Non–structural elements and attachments must follow the deformation of the structure.

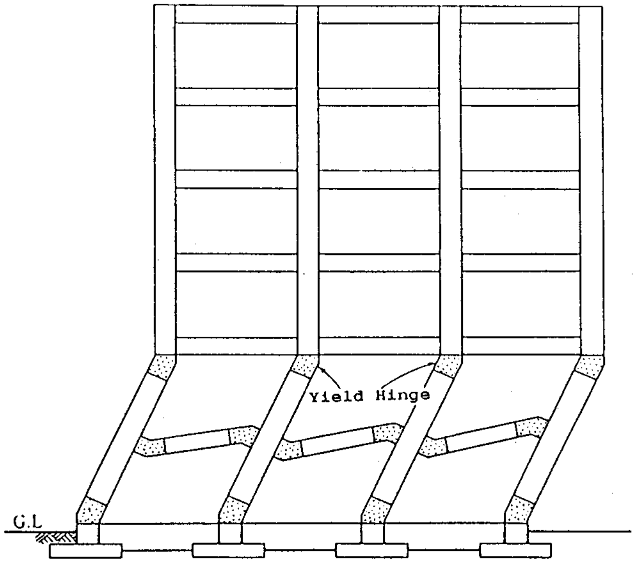


Fig. 3: Partial Yield Mechanism

STRUCTURAL ANALYSIS METHOD

A complete history of member and structural resistance with deformation must be obtained for a structure, as designed, under monotonically increasing lateral loads. The analysis may be terminated when a maximum story drift at a story reaches the Proof Deformation. The results of an analysis may, however, be significantly affected by the modeling of a structure and its members. Therefore, the method of modeling and the resistance–deformation relation of members are outlined in the commentary of the guidelines.

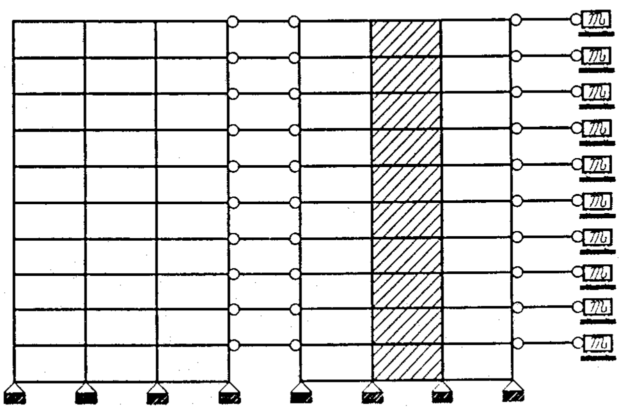


Fig. 4: Member Modeling of Plane Frames

Structural Modeling -- A structure must, as a general rule, be analyzed including the foundation structure, idealized as a series of plane frames inter-connected by rigid truss members at floor levels (Fig. 4). Three-dimensional effect of a structural wall (the contribution of girders orthogonal to the wall to the vertical movement at the wall boundary columns) must be included in the analysis (Fig. 5). The structure may be analyzed separately in the two principal directions if the effect of torsion and transverse frames can be neglected.

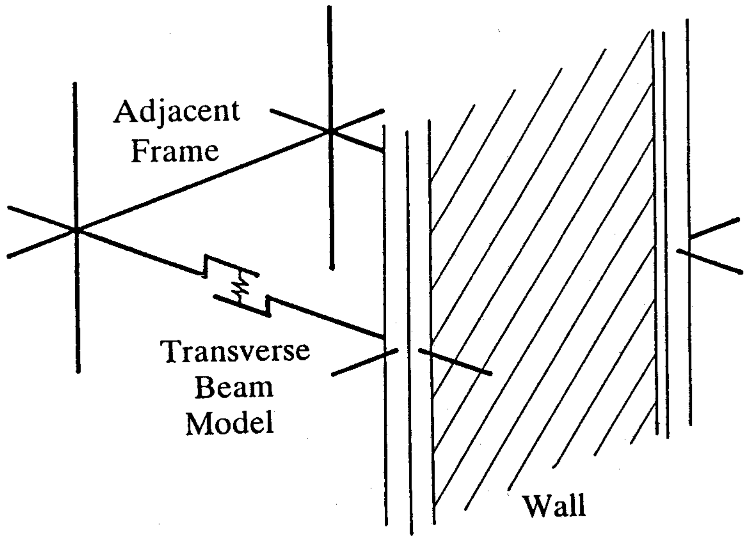
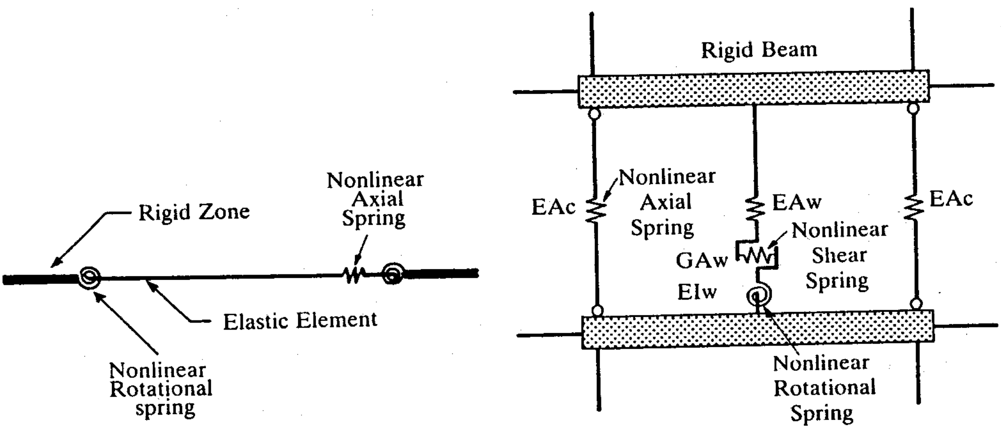


Fig. 5: Modeling of Structural Wall with Connecting Girders



(a) Beams and Columns (b) Structural Walls

Fig. 6: Member Model

Modeling of Structural Members -- Inelastic deflection of a column and beam may be assumed to concentrate at the member ends, represented by rotation of rigid–plastic rotational springs (Fig. 6). The stiffness characteristics of a column and beam under monotonically increasing load may be assumed to be of trilinear type with stiffness changes at flexural cracking and yielding. A beam–column connection may be assumed to be rigid.

Flexural yield moment at specified yield hinges may be calculated using 1.1 time the specified material strength for the longitudinal reinforcement because the actual strength of the reinforcing steel available in Japan is shown to exhibit at least 1.1 times the specified yield strength.

Loads -- A structure must be analyzed under dead loads and live loads specified for gravity loading and under monotonically increasing lateral loads distributed in a pattern the same as the design earthquake loading for serviceability limit state.

SERVICEABILITY LIMIT STATE DESIGN

Design Earthquake Loading -- It is desirable to define the intensity and characteristics of earthquake motions expected at each construction site, and to perform realistic nonlinear earthquake response analyses of a structure to examine the safety. The intensity in terms of maximum ground velocity may be generally considered to be approximately 150–200 mm/sec. Due to the lack of reliable earthquake information about intensity and characteristics at present, however, the characteristics of a medium intensity ground motion for use in the serviceability limit state design has not been clearly defined. Therefore, the PRESSS guidelines adopts the design earthquake load level equal to that of the allowable stress design (standard base shear coefficient of 0.2) defined in the Law Enforcement Order (Building Center of Japan, 1990). The level of safety is intended to maintain the same as the existing building.

Design earthquake story shear Q_i is calculated by multiplying the total of dead and live loads (reduced for earthquake loading) W_i at and above story i by a seismic story shear coefficient C_i at the story:

$$Q_i = C_i W_i \tag{3}$$

$$C_i = Z R_t A_i C_B \tag{4}$$

where, Z: a seismic zone factor (= 0.7 to 1.0, Fig. 7), R_t: a vibration characteristic factor of a building taking into account the type of soil (Fig. 8), A_i: a factor representing vertical distribution of seismic story shear coefficient (Fig. 9), C_B: standard base shear coefficient.

Vibration characteristic factor R_t is given as follows:

$$R_t = 1.0 \qquad \text{for} \qquad T < T_c \tag{5.1}$$

$$R_t = 1.0 - 0.2\,(T / T_c - 1)^2 \qquad \text{for} \qquad T_c < T < 2T_c \tag{5.2}$$

$$R_t = 1.6\,(T_c / T) \qquad \text{for} \qquad 2T_c < T \tag{5.3}$$

where, T_c: critical period of subsoil (0.4 sec for stiff sand or gravel, 0.6 sec for other soil, and 0.8 sec for alluvium mainly consisting of organic or other soft soil), T: period of a building, calculated by

$$T = 0.02\, h \tag{6}$$

where, h: total height of the building in m.

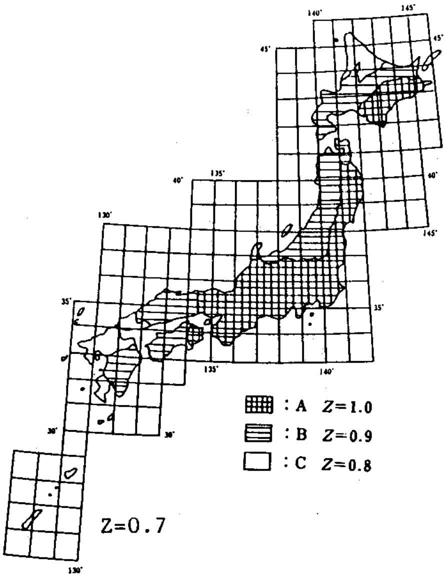


Fig. 7: Seismic Zone Factor

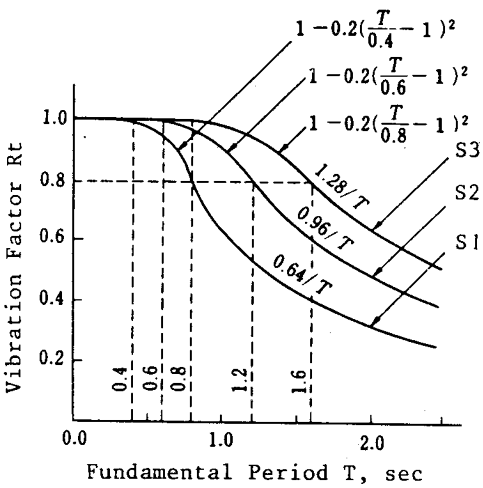


Fig. 8: Vibration Characteristic Factor

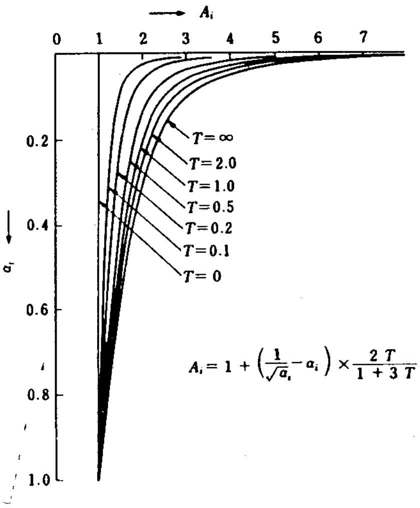


Fig. 9: Story Shear Distribution Factor

The coefficient A_i is given by the following expression:

$$A_i = 1 + (1/\sqrt{\alpha_i} - \alpha_i)[2\,T / (1 + 3\,T)] \qquad (7)$$

where,

$$\alpha_i = W_i / W_1 \qquad (8)$$

The standard base shear coefficient C_B is 0.2 for the serviceability limit state. The standard base shear coefficient may be used to express the lateral load carrying capacity of a structure in the ultimate limit state design.

Design Criteria -- At the design earthquake load, a building must not develop flexural yielding at any member end, and the story drift angle (inter-story drift divided by story height) must be less than 1/200 rad, the value of which is intended to protect non-structural elements. The story drift must be calculated by nonlinear static analysis based on realistic stiffness properties of constituent members under monotonically increasing lateral loading. The drift limit requirement often governs the design.

ULTIMATE LIMIT STATE DESIGN

A dynamic response amplitude is closely related to the lateral strength. Hence, the required lateral load resisting capacity should ideally be determined so that the maximum response of a structure during design earthquake motions should be less than the design limit deformation. The intensity of the design earthquake motion may be generally considered to be approximately 400 - 500 mm/sec in terms of maximum ground velocity, but the detailed characteristics of the design earthquake motion are not clearly defined due to the lack of information about the characteristics of earthquake motions at unspecified construction site. Therefore, the PRESSS guidelines adopts the design earthquake load level equal to that used in the examination for the ultimate lateral load resisting capacity defined in the Law Enforcement Order (Building Center of Japan, 1990).

Required Lateral Load Resisting Capacity -- A structure must develop a lateral load resistance larger than 90 percent of the required lateral load resisting capacity, defined by Eqs. (3) and (4), when a story drift angle at any story reaches the Limit Deformation (drift angle R_{u1}), and that larger than the required lateral load resisting capacity at the Proof Deformation (drift angle R_{u2}). The lateral load resisting capacity is expressed in terms of base shear coefficient C_B, and is consistent with that required for a ductile structure by the Law Enforcement Order (Building Center of Japan).

The values of R_{u1}, R_{u2} and C_B are listed in Table 1, in which the values are varied with the ratio of base overturning moment resisted by the structural walls at R_{u1}. The required base shear coefficient varied from 0.3 to 0.4, and the design limit story drift angle from 1/100 to 1/150 rad (Fig. 10). These values were selected recognizing that structural walls enhance the stiffness and lateral load resistance of a structure, but that the structural walls reduce the deformation at the maximum resistance. The values of Limit Deformation do not have any logical ground, but the drift limitation of 1/100 rad is commonly used in the design of high-rise moment-resisting frame buildings.

Regions, where yielding is permitted in design, must be provided with ductility to a deformation amplitude calculated by the nonlinear static analysis at the Proof Deformation.

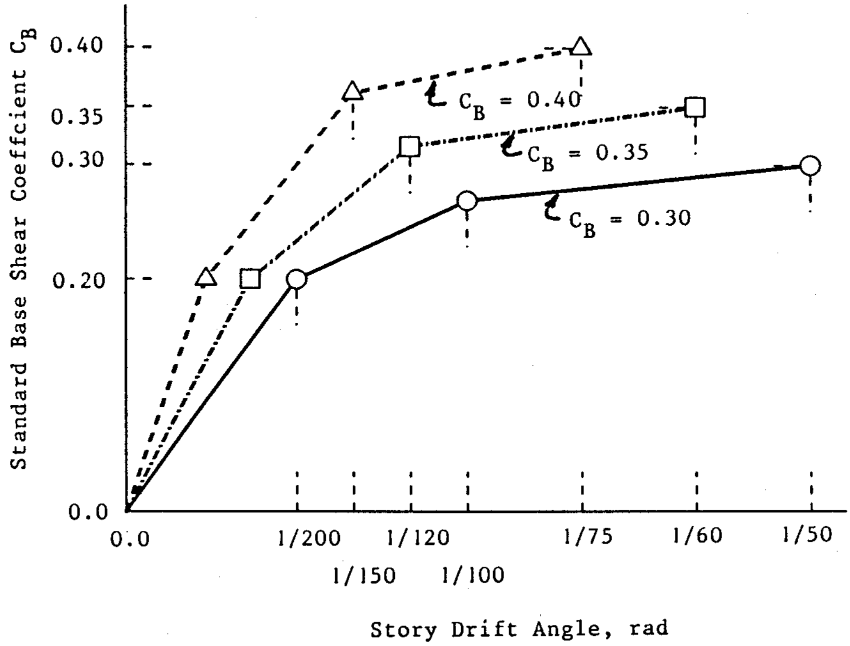


Fig. 10: Performance Criteria for Frame–Wall Structures

Required Member Strength -- In order to assure the formation of a specified yield mechanism of a structure during a strong earthquake, regions other than the acceptable yield hinge locations must be provided with resistances against all modes of failure, such as flexural yielding, shear failure and bond failure, even under the upper bound actions during the strong earthquake motion. The vibration of a structure, forming a total yield mechanism, is normally dominated by the fundamental mode. Thus, basic required member strength is calculated by a static nonlinear analysis at the Proof Deformation, and then multiplied by corresponding amplification factors to yield the required member strength.

The amplification factors must take into consideration the followings;
(a) increase in material strength above the specified value,
(b) lateral load distribution during an earthquake excitation different from the one assumed in the nonlinear static analysis,
(c) bi-directional earthquake loading,
(d) increase in slab width effective to girder flexural resistance,
(e) increase in material strength due to strain hardening,
(f) reliability of structural analysis methods,
(g) reliability of member strength estimate, and
(h) reliability of workmanship.

The first three terms are considered in the PRESSS guidelines. The expected plastic deformation is believed to be small at specified yield hinges in the building designed in accordance with the guidelines, hence, the increase in member resistance due to the increase in effective slab width and the increase in material resistance due to strain hardening was not considered.

Amplification by Material Strength -- According to a report (Takahashi, 1985), the compressive strength of concrete fell below the design nominal strength in less than 0.5 % of standard water-cured coupons obtained at construction sites (Fig. 11). It is known that the concrete strength does not influence the flexural strength of a girder. On the other hand, flexural strength may be affected by the concrete strength in a column subjected to high axial load, but the number of yield hinges in such columns are limited. Therefore, the design specified strength may be used for concrete strength in evaluating the flexural strength at a specified yield hinge.

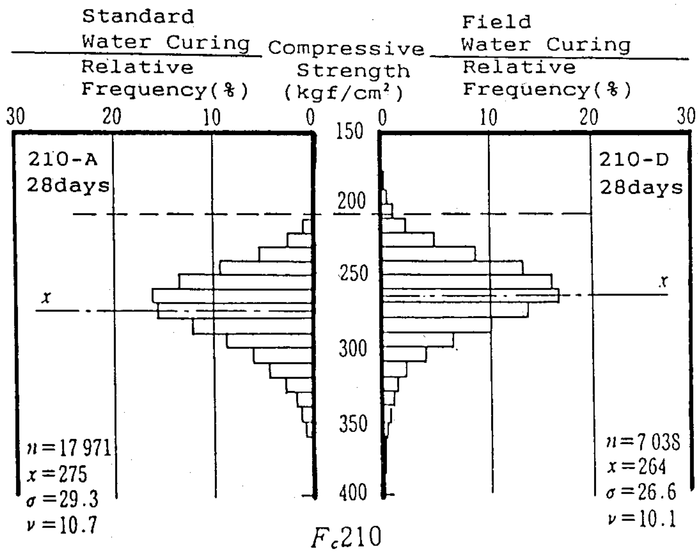


Fig. 11: Statistical Distribution of Concrete Strength

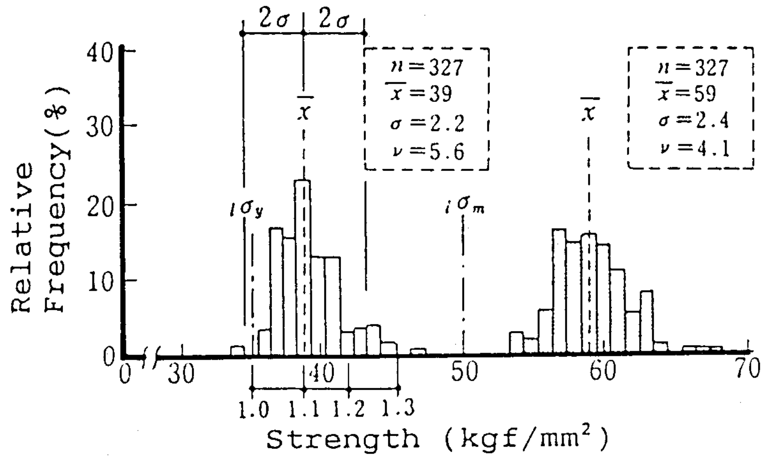


Fig. 12: Statistical Distribution of Yield Strength in Reinforcing Steel

Ikeda (1986) reported the results of tension tests of reinforcing bars, in which the lower limit yield stress for Grade SD295 steel was approximately 1.1 times the specified yield strength, and those for Grades SD345 and SD390 steel were comparable to the specified values. The upper bound yield stress (statistical average plus twice the standard deviation) may reach as high as 1.30 times the specified yield strength for Grade 295 steel, and 1.25 times the specified yield strength for Grades 345 and 390 steel (Fig. 12). Steel grades SD345 and SD390 are mostly used as longitudinal reinforcement.

The use of 1.1 times the specified yield strength is permitted in the calculation of flexural resistance in design (Building Center of Japan, 1990). Therefore, flexural resistance at a yield hinge may be increased by approximately 1.1 (=1.25/1.1) if the longitudinal reinforcement exhibits the upper bound strength. For example, the basic shear for a girder, normally calculated for yielding of the girder at both ends, must be amplified by an amplification factor of 1.1 to estimate the maximum possible shear force in the girder.

Amplification by Dynamic Effect -- The lateral load distribution during an earthquake is different from those assumed in a static analysis, the phenomenon which is called dynamic effect; i.e., member actions fluctuate from the basic design forces obtained by the static analysis. On the basis of a series of nonlinear earthquake response analysis of frame and wall-frame structures, Kabeyasawa (1985) reported that maximum higher mode forces during a nonlinear earthquake response were proportional to the intensity of input motion and mass distribution of the structure. The story shears associated with the fundamental mode are limited by the static story shears at the formation of a total yield mechanism; hence the dynamic effect increases with the degree of plastic deformation of a structure. Larger actions tend to be attracted to stiffer vertical members (AIJ, 1990).

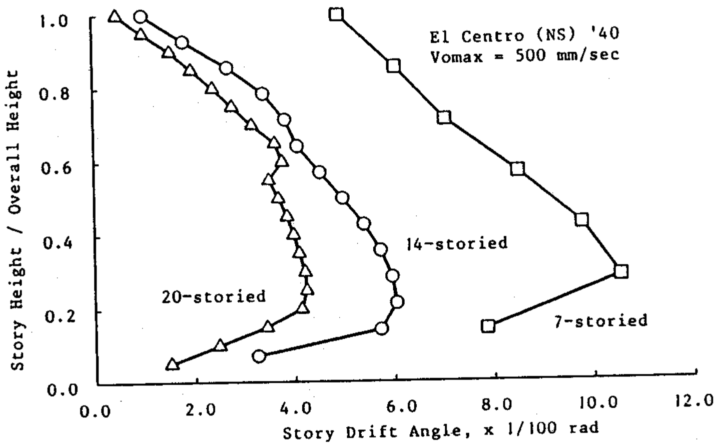
Although a shear distribution among columns of a story may vary with time during an earthquake, the dynamic amplification factor for story shears is used for column shears. Column end moments fluctuate more during an earthquake attributable to the shift of an inflection point. Although higher mode oscillation might cause instantaneous yielding at an end of each column, a partial story yield mechanism will not be formed because the story shear is limited. A dynamic amplification factor for column moments is comparable to the dynamic amplification factor for a story shear.

In a wall-frame structures, a wall has a tendency to carry most of the higher mode shears because the wall is hard to deform in a higher mode shape. Hence, larger higher mode story shears are assigned to a wall. The dynamic amplification factor, derived for story shear forces, does not apply well to moments in a structural wall. However, flexural capacity of a structural wall does not require the same safety margin as that of columns because a story yield mechanism will not form in a structural wall if shear failure is prevented. Therefore, a constant dynamic amplification factor is used in a wall.

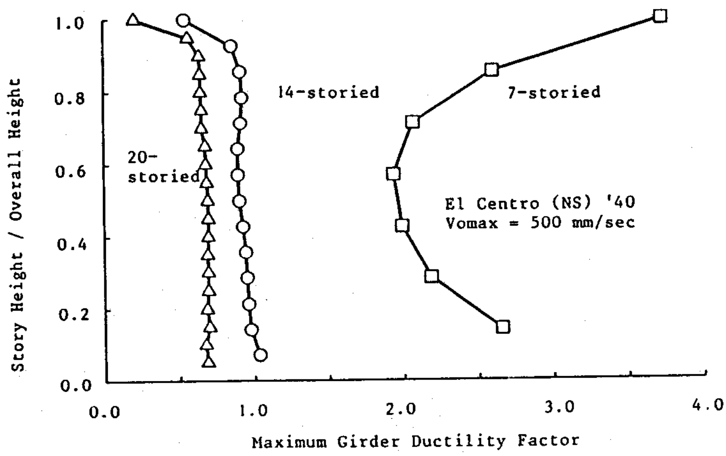
A series of simple structures of 5-story (15 m tall) to 20-story (60 m tall) buildings were analyzed under design earthquake motions, and maximum column actions were calculated. Dynamic amplification factors were determined as ratios of maximum column actions (basic member forces) to the corresponding column actions calculated at the Proof Deformation, which is much beyond expected response by a design earthquake motion. For a column moment, the larger of the column top and bottom moments was used at a floor. External columns tend to attract larger earthquake forces than interior columns.

Following the general practice in design of a building taller than 60 m, the intensity of earthquake motions for a response analysis was selected to be 500 mm/sec in maximum ground velocity. El Centro (NS) 1940 record (amax = 5.0 m/sec2), Taft (EW) 1952 record (amax= 5.2 m/sec2), Tohoku University (NS) 1978 record (amax= 3.7 m/sec2) and Hachinohe Harbor (EW) 1968 record (amax= 2.4 m/sec2), were used in nonlinear earthquake response analyses.

According to the results of nonlinear earthquake response analyses of simple buildings, designed in conformance with the Building Standard Law (Building Center of Japan, 1990), the response story drift and ductility demand decrease with the structural height (Fig. 13); i.e., the lateral load resisting capacity specified in the Building Standard Law includes a larger safety margin for a taller structure. By the same token, a structure, provided with lateral load resistance larger than that specified in the Building Standard Law, develops maximum story drift smaller than the design limit deformation. Similarly, the design limit deformation is not reduced for a tall structure.



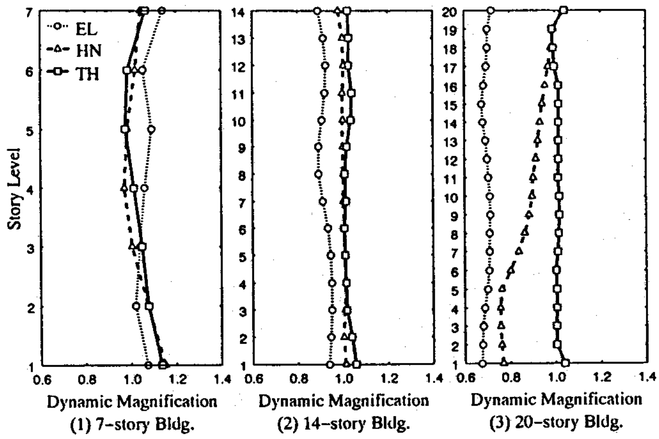
(a) Maximum Story Drift Angle



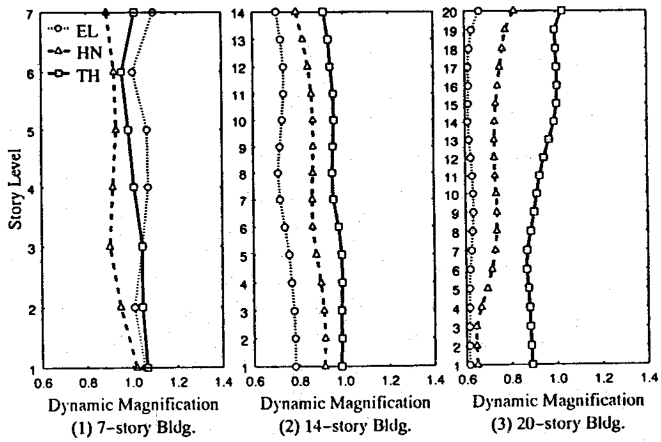
(b) Maximum Girder Ductility Demand

Fig. 13: Earthquake Response and Building Height

Dynamic amplification factor, defined as a ratio of maximum response column shear to the column shear calculated by the nonlinear static analysis at the Proof Deformation, was calculated for representative bench mark structures under four earthquake motions (El Centro NS 1940, Taft EW 1952, Tohoku University NS 1978, and Hachinohe Harbor EW 1968 records) normalized for the maximum ground velocity of 500 mm/sec. The dynamic amplification factor was 1.1 to 1.2 for seven-story building, and 1.0 for 14-story and 20-story structures (Fig. 14).



(a) Frame Building with Standard Base Shear Coefficient of 0.30



(b) Frame Building with Standard Base Shear Coefficient of 0.40
Fig. 14: Dynamic Amplification Factor

Amplification by Bi-directional Earthquake Loading -- A structure is subjected to horizontal bi-directional and vertical motion in an earthquake although the structure is normally designed separately and independently in the two principal directions. Exterior and corner columns of a frame are subjected to varying axial load due to an earthquake overturning moment acting on the structure in addition to bi-directional lateral load reversals. The interaction of resistance among axial load and bi-directional moments is significant in a reinforced concrete column. For a constant axial load, a yield surface of a column under bi-directional bending forms a circular shape; i.e., the lateral load resistance is almost constant in all direction.

As a beam resists an earthquake load only in one direction, the resistance of a beam yield-type structure is not affected by the earthquake load in the orthogonal direction. Therefore, if a structure forms a beam yield-type total collapse mechanism simultaneously in the two principal directions, columns at each story are subjected to 1.41 times the lateral forces at the formation of the collapse mechanism in one direction. Consequently, unless additional resistance is provided in a column, the column may fail when a structure is subjected to a strong bi-directional earthquake motion. To assure the beam yield-type mechanism under a bi-directional earthquake, the columns should be designed considering simultaneous formation of collapse mechanism in the two orthogonal directions. However, the probability of concurrent mechanism formation in the two directions is small.

In the development of the PRESSS guidelines, the uni-axial design earthquake loading is assumed to act in any direction. For simplicity and discussion purpose, the followings were assumed:

(a) yielding takes place simultaneously at all girder ends and at the base of first-story columns in a moment-resisting frame structure under monotonically increasing lateral loading,

(b) the member yield resistance is identical in the two orthogonal directions,

(c) maximum response deformation during a design earthquake motion is comparable in any direction, and

(d) the yield surface of a column under bi-directional loading is approximated by a circle.

A simple 10-story bench mark structure was analyzed under uni-directional motion in the principal direction and 45-degree direction in plan, and under bi-directional earthquake motion (Fig. 15). The maximum response of the structure under the 45-degree uni-directional motion was calculated as a vectorial sum of the response amplitudes in the two principal direction at each moment. The maximum response amplitudes are comparable for uni-directional earthquake motion in a principal direction or in the 45-degree direction.

The lateral resistance in a principal direction is governed by the flexural resistance of girders, which is not influenced by the loading in the orthogonal direction (Fig. 16). Under loading in the diagonal direction, yielding resistance and deformation become 1.4 times larger than the corresponding resistance and deformation in a principal direction.

If yielding does not take place in a structure (Region 1 in Fig. 16) during a principal loading, then the diagonal loading will not develop column moments and shears larger than those at the yielding in the principal direction; i.e., the design moment and shear of columns need not be amplified for diagonal loading. On the other hand, if a large plastic deformation takes place in a structure (Region 3 in Fig. 16) during a principal loading, the diagonal loading is likely to develop simultaneous yielding in both principal directions; i.e., the design moment and shear of columns must be amplified by 1.4 from those under principal loading to prevent column flexural yielding or shear failure.

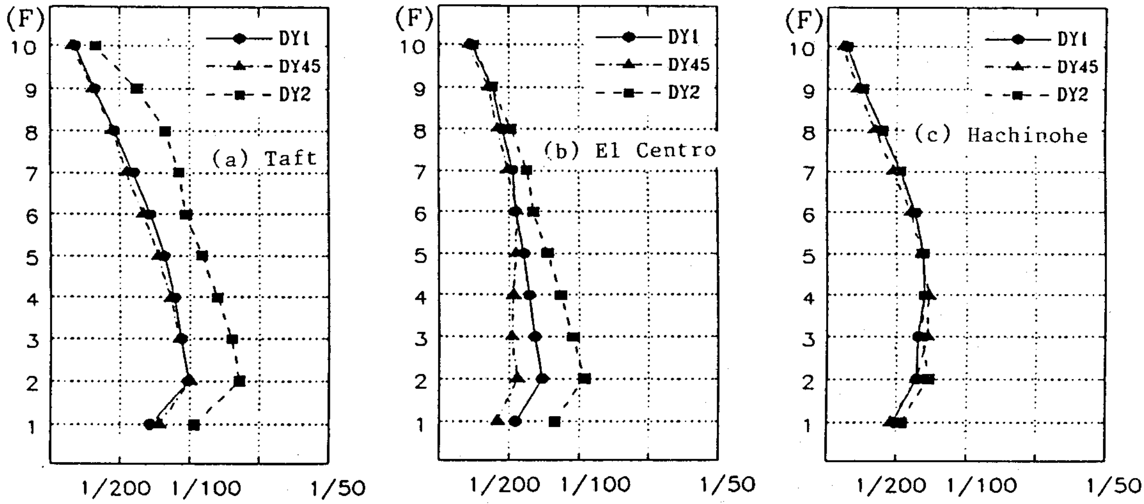


Fig. 15: Maximum Response under Uni-directional Earthquake Motion

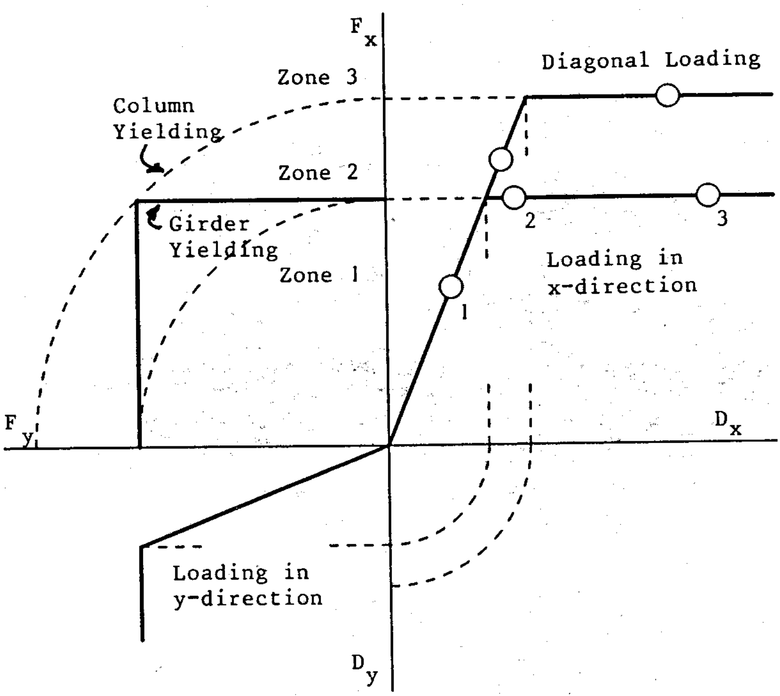


Fig. 16: Effect of Bi-directional Loading

Therefore, the basic moments and shears of a column must be amplified by a factor between 1.0 to 1.4, depending on the plastic deformation estimated by the nonlinear dynamic analysis under a design earthquake motion in the principal direction.

Note that the effect of bi-directional loading and the dynamic amplification increase with the ductility demand. Therefore, the amplification factor for member design forces is reduced for a structure with a larger safety margin (a tall or strong structure). The amplification factors are listed in Table 2, and expected response regions in Table 3.

Axial Load Limitation -- Axial force in columns must be less than 2/3 N_u and 3/4 N_t, where N_u: ultimate compressive strength of the column, and N_t: ultimate tensile strength.

FOUNDATION STRUCTURES

Foundation and basement must be provided with rigidity and strength against gravity loading, during a strong wind, snow loading and medium intensity earthquake motions, sufficient to prevent excessive settlement, inclination and sliding for serviceability, and to prevent excessive cracking for durability.

Design stresses for basement structures include the effect of soil pressure and hydraulic pressure in addition to the effect of long-term gravity load and short-term snow, wind pressure and earthquake loads transmitted from the super-structure. Stresses caused by uneven settlement and lifting or deflection of the soil and piles must be considered if appropriate.

Stresses at an earthquake load level for the serviceability limit state of the superstructure must not exceed the allowable stresses for the short-term loading; furthermore, the foundation must not be lifted. At an earthquake load level for the ultimate limit state of the super-structure, forces in the foundation structure must not exceed the ultimate strength of members and soil.

Foundation structure, such as foundation girders, slab and piles, must be planned not to form a part of the specified yield mechanism. The properties of sub-structures are not clearly understood in relation to the behavior of super-structures. The sub-structure must be protected because it is often difficult to investigate the damage in the foundation after an earthquake and because the cost to repair the foundation damage is enormous.

ACKNOWLEDGEMENT

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Table 1: Required Base Shear Coefficient C_B, Limit Drift Angle R_{u1} and Proof Drift Angle R_{u2}

Structural Type	CB	R_{u1}(rad)	R_{u2}(rad)
$\beta < 0.3$	0.3	1/100	1/50
$0.3 < \beta < 0.7$	0.35	1/120	1/60
$0.7 < \beta$	0.4	1/150	1/75

β: ratio of base overturning moment resisted by structural walls at R_{u1}

Table 2: Amplification Factors for Design Forces

Members	Actions	Expected Response		
		Region 1	Region 2	Region 3
(a) Girders	Shear	1.1	1.1	1.1
(b) Interior	Moment	1.1	1.3	1.5
Columns	Shear	1.1	1.3	1.5
(c) Exterior	Moment	1.3	1.5	1.5
Columns	Shear	1.3	1.5	1.5
(d) Structural	Moment	1.2	1.2	1.2
Walls	Shear	1.2	1.2	1.2
(e) Beam–column Connection	Shear	1.1	1.3	1.5

Table 3: Structural Height, Lateral Load Resisting Capacity and Expected Response for Frame Structures

Base Shear Coeff., C	Structural Height, H		
	H<20 m	20 m<H<45 m	45 m<H<60 m
0.27 < C <0.315	Region 3	Region 2	Region 1
0.315< C <0.36	Region 3	Region 2	Region 1
0.36 < C	Region 2	Region 1	Region 1

ULTIMATE STRENGTH DESIGN GUIDELINES

FOR

REINFORCED CONCRETE BUILDINGS

by

PRESSS GUIDELINES WORKING GROUP

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March 31, 1993

ABSTRACT

This paper introduces "Ultimate Strength Design Guidelines for Reinforced Concrete Buildings," developed as a part of U.S.–Japanese Coordinated PRESSS (Precast Seismic Structural System) project. The guidelines has been drafted by Guidelines Drafting Working Group, and discussed in Design Guidelines Committee of Japanese PRESSS. Extensive commentary will accompany the guidelines to explain the concept behind requirements and to suggest methods of calculation.

LIST OF CONTENTS

CHAPTER 1 SCOPE

1.1 General Requirements

(1) This design guidelines provides minimum requirements for the design of cast–in–situ reinforced concrete (RC) buildings and reinforced concrete (PCaRC) buildings which use precast reinforced concrete (PCa) members for earthquake resistant elements.

(2) The application of some requirements can be exempted if a structure, designed on the basis of special studies, can be demonstrated to posses structural performance as good as or superior to those buildings designed under this guidelines.

1.2 Building Height

The total height of a building shall be not more than 60 m.

1.3 Structural System

A structure, in each principal direction, shall consist of ductile moment–resisting frames, or of ductile moment–resisting frames combined with continuous structural walls, or of independent structural walls. The structural wall shall be continuous from the foundation to the roof level.

1.4 Structural Configuration

(1) Eccentricity ratio and rigidity ratio of a structure, defined in Article 82–3 of Building Standard Law Enforcement Order, shall be not more than 0.15 and not less than 0.6, respectively.

(2) Height–to–width ratio (aspect ratio) at any level of a building shall be, as a general rule, not more than 4.

1.5 Yield Mechanism

The structure of a building shall be clearly planned to form a specified total yield mechanism, in which flexural yield hinges shall develop, as a general rule, at the ends of all floor beams and at the base of the first story columns and structural walls.

1.6 Site Geology

Soil types at a construction site shall be, as a general rule, Type 1 or Type 2, defined in Notification No. 1793 of Ministry of Construction.

1.7 Definitions

Some technical terms, used specifically in the guidelines, are defined below;

PCa connection: Connection between two PCa elements or between a PCa element and an

RC member;

Major earthquake resisting elements: Out of major structural parts, defined in Article 3–1 of Building Standard Law Enforcement Order, columns, girders, structural walls, foundation girders, foundation slabs and foundation piles;

Yield hinge (region): The location (region) to develop plastic deformation by flexural yielding under the action of bending;

Special yield hinge: Yield hinge of columns and structural walls where special confining reinforcement is required to resist high axial loading;

Non–yield hinge (region): The location (region) where yield hinge does not form;

Serviceability limit state design: Design of a structure under long–term loading and small to medium intensity earthquake motion to ensure serviceability of the structure;

Ultimate limit state design: Design of a structure under a strong earthquake motion, that may occur once during the life time, to ensure the safety and reuse of the structure with extensive repair work;

Design limit deformation R_{u1}: Deformation of a structure or members expected to develop under an intense earthquake motion;

Design proof deformation R_{u2}: Deformation of a structure to which the performance of the structure is assured in design.

CHAPTER 2 DESIGN PRINCIPLES

2.1 Structural Performance

2.1.1 Building

(1) A building shall resist gravity loads and medium intensity ground motions, wind pressure and snow loads without disturbing serviceability.

(2) The structural part of a building above the ground level shall be designed to develop total yield mechanism under an intense earthquake motion, and shall be provided with stiffness and lateral resistance sufficient to limit the lateral deformation within a specified value.

Foundation structure and the structural part in the basement shall safely transfer the vertical and lateral loads from the super-structure to the soil, and major earthquake resisting elements in the basement, foundation girders, foundation slabs and piles shall not, as a general rule, yield even under an intense earthquake motion.

2.1.2 PCa Members and Connections

(1) PCa connections shall be provided with strength sufficient to transfer member actions caused by specified design loads.

(2) PCa connections shall be designed to limit the strength deterioration and slippage deformation, inherent to PCa connections, to satisfy "Evaluation Criteria for PCa Connection Performance".

(3) PCa members and connections shall satisfy required serviceability, durability and fire resistance.

2.1.3 Non-structural Elements

(1) Non-structural elements shall be connected to structural members to ensure serviceability during gravity, snow loading, wind pressure, and during medium intensity earthquake motions, and also not to influence the development of the specified yield mechanism of a structure during an intense earthquake motion.

(2) Non-structural elements and attachments shall be fastened to structural members so that their falls will not damage the function and safety of the building.

2.2 Method of Structural Design

2.2.1 Design Principle

Design of major structural members of a building shall be designed for gravity loads, earthquake loads, wind pressure and snow loads to satisfy the structural performance defined in Chapter 1 and Section 2.1. Structural calculation of PCa members may follow the method for RC members.

2.2.2 Design for Gravity Loads

(1) Stress in every part of major structural members under dead load, specified in Article 84 of Building Standard Law Enforcement Order, live load, specified in Article 85, and snow load, specified in Article 86, in heavy snow zones designated by specific administrative office, shall not exceed allowable stresses for the long-term loading specified in Section 3.3.

Long term loads shall include soil pressure, water pressure, vibration, impact, temperature, shrinkage, uneven ground settlement, if applicable, to the structure.

(2) Structural members and PCa connections shall not develop excessive cracking, deflection or vibration for serviceability and durability under the loads defined above.

2.2.3 Design for Earthquake Loads

(1) Serviceability and ultimate limit state design of structural members in the super-structure under earthquake loads shall conform to the provisions of Chapter 4. Those buildings not taller than 31 m can be designed by the provisions of Chapter 5.

(2) Serviceability and ultimate limit state design of structural members in the basement and foundation under earthquake loads shall conform to the provisions of Chapter 6.

2.2.4 Design for Wind Pressure

Stress in every part of structural members under wind pressure and combined loads, stipulated in Paragraph 2 of Article 82-1 of Building Standard Law Enforcement Order, shall not exceed allowable stresses for the short-term loading specified in Section 3.3.

2.2.5 Design for Snow Loads

Stress in every part of structural members under snow loads and combined loads, stipulated in Paragraph 2 of Article 82-1 of Building Standard Law Enforcement Order, shall not exceed allowable stresses for the short-term loading specified in Section 3.3.

CHAPTER 3 MATERIALS

3.1 Quality and Type

(1) Concrete for RC Members

(a) Types by aggregate and specified compressive strength of concrete for RC members shall conform to Table 3.1.

Table 3.1: Types and Specified Strength of Concrete for RC and PCa Members

Types of Concrete	Specified Strength (kgf/cm^2)
Normal Concrete	210 – 360
Light-weight Concrete	210 – 270

(b) Quality, mix, production, materials of concrete for RC members shall meet the provisions of Japan Architectural Standard Specification JASS-5 "Reinforced Concrete Work," published by Architectural Institute of Japan.

(2) Concrete for PCa Members

(a) Types aggregate and specified compressive strength of concrete for PCa members shall conform to Table 3.1.

(b) Quality, mix, production, materials of concrete for PCa members shall meet the provisions of JASS-5.

(3) Concrete for PCa Connection

(a) Types of concrete for PCa connection shall be as good as or superior to those of concrete for RC members.

(b) Quality, mix, production and materials for PCa connection shall meet the provisions of "PRESSS PCa Construction Guidelines."

(4) Reinforcing Bars and PC Steel Bars

(a) Reinforcing bars shall meet the Japan Industrial Standards (JIS) G-3112 "Steel Bars for Concrete Reinforced."

Round Bar	SR235, SR295
Deformed Bar	SD295A, SD295B, SD345, SD390
Nominal Diameter	D10 to D41 and π 9 to π 13

(b) Prestressed concrete steel bars shall meet the provisions of JIS G-3109 "Steel Bar for Prestressed Concrete." Prestressing wire and strand shall meet the provisions of JIS G-3536

"Uncoated Stress-Relieved Steel Wires and Strand for Prestressed Concrete."

(c) Welded wire fabric shall meet the provisions of JIS G-3551 "Welded Steel Wire Fabric." Nominal diameter of steel wire shall be not less than 4 mm.

(5) Grout and Mortar

(a) Compressive strength of mortar used at PCa connections shall be as high as or superior to that of concrete for RC and PCa members.

(b) Types, quality, mix, production and materials shall meet the provisions of "PRESSS PCa Construction Guidelines."

(6) Steel Materials

Quality of steel elements for PCa members shall be specified in design specifications. The shape and dimensions of steel elements shall be specified in design specifications and drawings.

(7) Joint of Reinforcing Bars and Steel

(a) Reinforcing bars may be jointed by gas pressured welding, flare welding, or lap splicing.

(b) Lap splice shall meet the requirements of "AIJ Standards for Structural Calculation of Reinforced Concrete Structures."

(c) Work of gas pressured welding shall meet "Standard Specification for Gas Pressured Welding Work for Reinforcing Bars" by Japan Gas Pressured Welding Institute.

(d) Steel plates may be jointed by welding or high tension bolts.

(e) Work of welding and high tension bolt friction joint of steel plates and work of flare welding of reinforcing bars shall meet the provisions of JASS-10 "Precast Concrete Work" and JASS-6 "Steel Work."

3.2 Material Constants

(1) Concrete

Material constants for mechanical properties of concrete for RC, PCa members and PCa connections shall be as follows:

Young's modulus: $2.1 \times 10^5 (r/2.3)^{1.5} (Fc/200)^{0.5}$ (3.1)

Poison's ratio: 1/6 (3.2)

Coefficient of thermal expansion: 1×10^{-5} (/deg C) (3.3)

where, r: weight per unit volume (tonf/m^3), and Fc: specified concrete compressive strength (kgf/cm^2).

(2) Reinforcing Bars

Material constants for mechanical properties of reinforcing bars shall be as follows:

Young's modulus: 2.1×10^6 kgf/cm^2 (3.4)

Coefficient of thermal expansion: 1×10^{-5} (/deg C) (3.5)

(3) Mortar and Grout

Young's modulus of mortar shall be assumed equal to the smaller value specified for RC and PCa members at the PCa connection.

(4) Steel

Material constants for mechanical properties of steel shall be as follows:

Young's modulus: 2.1×10^6 kgf/cm^2 (3.6)

Coefficient of thermal expansion: 1×10^{-5} (/deg C) (3.7)

3.3 Allowable Stresses and Material Strength

(1) Concrete

Allowable stresses and material strength of concrete shall be taken as specified in Tables 3.2 and 3.3. Material strength for bond may be determined by experimental or analytical study.

(2) Reinforcing Bars

Allowable stresses and material strength of reinforcing bars shall be taken from Table 3.4.

(3) PC Steel Bars

Allowable stresses and material strength of prestressed concrete steel bars, steel wire and strand shall be equal to the values specified in "AIJ Standard for Design and Construction of Pre-stressed Concrete."

(4) Steel

(a) Allowable stress of steel shall be as specified in Article 90 of Building Standard Law Enforcement Order.

(b) Material strength of steel shall be as specified in Article 96 of Building Standard Law Enforcement Order.

(5) Mortar and Grout

Allowable stresses and material strength of mortar and grout shall be equal to the smaller value of RC and PCa members.

Table 3.2 : Allowable Stresses and Material Strength for Concrete (kgf/cm^2)

Loading Type	Action	Normal Concrete	Light-weight Concrete
Long-term Loading	Compression	Fc / 3	Fc / 3
	Tension	------	------
	Shear	Fc / 30 and (5 + Fc / 100)	0.9 times the value for normal concrete
Short-term Loading	Compression	2 times the values for long-term loading	
	Tension	------	------
	Shear	1.5 times the values for long-term loading	
Material Strength	Compression	Fc	Fc

Fc : Specified compressive strength of concrete

Table 3.3 : Allowable Bond Stresses between Reinforcement and Concrete (kgf/cm^2)

Bar Type	Long-term Loading: Top Reinforcement	Long-term Loading: Bottom Reinforcement	Short-term Loading
Round	0.04 Fc and 9.0	0.06 Fc and 13.5	1.5 times the values for long-term loading
Deformed	Fc / 15 and (9 + 2 Fc / 75)	Fc / 10 and (13.5 + Fc / 25)	

a) Top reinforcement: Horizontal reinforcement with more than 30 cm depth of concrete below in a flexural member;
b) For a deformed bar with concrete cover less than 1.5 time bar diameter, allowable bond stress shall be reduced by the ratio of cover depth to the length of 1.5 times bar diameter.

Table 3.4 : Allowable Stresses and Material Strength for Reinforcement (kgf/cm^2)

	Long-term Loading		Short-term Loading		Material Strength	
Steel Grade	Tension Compres.	Shear Reinf.	Tension Compres.	Shear Reinf.	Tension Compres.	Shear Reinf.
SR235	1,600	1,600	2,400	2,400	2,400 x 1.1	2,400
SR295	1,600	2,000	3,000	3,000	3,000 x 1.0	3,000
SD295A SD295B	2,000	2,000	3,000	3,000	3,000 x 1.1	3,000
SD345	2,200 (2,000)[1]	2,000	3,500	3,500	3,500 x 1.1	3,500[2]
SD390	2,200 (2,000)[1]	2,000	4,000	4,000	4,000 x 1.1	4,000[2]
Welded Wire	2,000	2,000	-----	3,000	3,000 x 1.1	----

Note: Values in parenthesis for deformed bars D29 and larger.

CHAPTER 4 EARTHQUAKE RESISTANT DESIGN (NONLINEAR ANALYSIS PROCEDURE)

4.1 Design Principles

4.1.1 Serviceability and Ultimate Limit State Design

The performance of super–structure of a building shall be examined for serviceability limit state under small to medium intensity earthquake motions and for ultimate limit state under an intense earthquake motion.

4.1.2 Method of Earthquake Resistant Design

(1) Earthquake resistant design shall be based on a static nonlinear analysis of a building under monotonically increasing lateral loading taking into account realistic elastic and inelastic characteristics of constituent structural members.

(2) The analysis shall be carried out in the longitudinal and transverse directions, separately.

(3) Lateral load shall be increased monotonically in the analysis under dead load, specified in Article 84 of Building Standard Law Enforcement Order, and live load for earthquake load calculation, specified in Article 85 of the order.

4.2 Serviceability Limit State Design

4.2.1 Design Earthquake Load

Design story shear Q_i at story i under the action of an earthquake motion shall be

$$Q_i = C_i\, W_i \tag{4.1}$$

$$C_i = Z\, R_t\, A_i\, C_B \tag{4.2}$$

where, C_i: story shear coefficient, W_i: sum of dead and live loads at and above the i–th story, Z: seismic zone coefficient, R_t: vibration characteristic coefficient, A_i: coefficient for story shear distribution, C_B: standard base shear coefficient of 0.2. Coefficients Z, R_t, and A_i are defined in Notification No. 1793 of Ministry of Construction, issued in 1980.

4.2.2 Earthquake Performance Criteria

The super–structure of the building shall satisfy the followings at the design earthquake load;

(1) No flexural yielding shall occur in structural members, and

(2) Story drift angle at each story shall be less than 1/200 rad.

4.3 Ultimate Limit State Design

4.3.1 Performance Criteria at Design Limit Deformation

Story shear resistance at each story, calculated at maximum story drift angle reaching design limit deformation, shall be greater than 90 percent of the required lateral load resisting capacity.

The required lateral load resisting capacity Q_{uni} of story i shall be

$$Q_{uni} = C_{uni}\, W_i \tag{4.3}$$

$$C_{uni} = Z\, R_t\, A_i\, C_{unB} \tag{4.4}$$

where, Z, R_t, W_i, and A_i are defined in Section 4.2.1. Design limit deformation R_{u1} and standard base shear coefficient for required lateral load resisting capacity C_{unB} are specified in Table 4.3.1.

Table 4.3.1: Standard base shear coefficient for required lateral load resisting capacity C_{unB}, design limit deformation R_{u1}, and design proof deformation R_{u2}

Ratio b_w of OTM resisted by walls	C_{unB}	R_{u1}	R_{u2}
$0.0 < b_w < 0.3$	0.30	1/100	1/50
$0.3 < b_w < 0.7$	0.35	1/120	1/60
$0.7 < b_w < 1.0$	0.40	1/150	1/75

where, b_w: the ratio of base overturning moment (OTM) resisted by structural walls at design limit deformation;

$$b_w = S\, {}_wQ_i\, H_i \,/\, (S\, Q_i\, H_i) \tag{4.5}$$

where, H_i: story height at i–th story, ${}_wQ_i$: sum of story shear carried by structural walls at i–th story, Q_i: total story shear at i–th story, S: sum from the first to the top story. ${}_wQ_i$ and Q_i are evaluated when the maximum story drift at a story reaches the design limit deformation.

4.3.2 Performance Criteria at Design Proof Deformation

The structure and its members shall satisfy the following conditions when the maximum story drift at a story reaches the design proof deformation R_{u2}, specified in Table 4.3.1;

(1) The story resistance at each story shall be greater than the required lateral load resisting capacity at the story,

(2) Flexural yielding shall not take place at the location and region where yield hinges are not permitted in Section 1.5,

(3) Shear and flexural strength of members, that yield at one or both ends at the design proof deformation, shall be greater than corresponding action of the member magnified by the amplification factors specified in Table 4.3.2.

(a) The bending moment and shear actions of a structural wall shall be taken from Figs. 4.3.1 and 4.3.2, respectively.

(b) Shear action in a girder yielding at one end shall be calculated by assuming simultaneous yielding at the two ends and gravity loads.

(c) Bending strength shall be examined at a member end where flexural yielding does not take place at the design proof deformation.

(d) Bond resistance of longitudinal reinforcement in a girder and column shall be examined for the reinforcement actually arranged in the member.

(4) Shear and flexural strength of members, that does not yield at either end at the design proof deformation, shall be greater than corresponding action of the member magnified by the amplification factors specified in Table 4.3.3.

(a) The bending moment and shear actions of a structural wall shall be taken from Figs. 4.3.1 and 4.3.2, respectively.

(b) Shear action in a girder yielding shall be calculated by assuming simultaneous yielding at the two ends and gravity loads.

(c) Bond resistance of longitudinal reinforcement in a girder and column shall be examined for the reinforcement actually arranged in the member.

(5) Limit of Column Axial Load

Column axial load, calculated for all earthquake loading directions, shall be greater than (3/4) Nt in tension, and less than (2/3) Nu, where,

$$Nu = Ac\ Fc \qquad (4.6)$$

$$Nt = Ag\ fy \qquad (4.7)$$

Ac: column cross sectional area, Fc: specified concrete strength, Ag: gross sectional area of column longitudinal reinforcement, and fy: material strength of column longitudinal reinforcement.

4.3.3 Design of PCa Connections

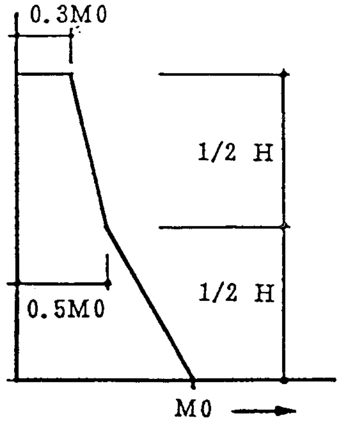
Design actions at a PCa connection shall be evaluated by the existing forces calculated at design proof deformation magnified by the amplification factors specified in Section 4.3.2. Design of PCa connections shall conform to the provisions of "Design Manual for PCa Connections."

Table 4.3.2: Amplification factors of design actions in member yielding at one or both ends

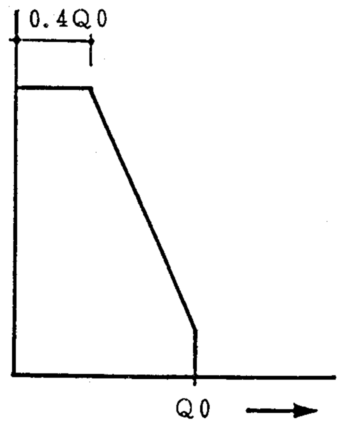
Member Action	Amplification Factor
(a) Girders	
Shear	a_1
(b) Columns	
Bending moment	a_2
Shear	a_3
(c) Structural Walls	
Bending moment	a_4
Shear	a_5

Table 4.3.3: Amplification factors of design actions in member without planned yielding

Member	Action	Amplification Factor
(a) Girders	Shear	a_1
(b) Columns	Bending	a_2
	Shear	a_3
(c) Walls	Bending	a_4
	Shear	a_5
(d) Beam–column Joints	Shear	a_6



Mo: Base bending moment calculated at design proof deformation;
H: Total height of a structural wall.
Fig. 4.3.1: Bending moment in a structural wall



Qo: Maximum story shear calculated at design proof deformation.
Fig. 4.3.2: Story shear in a structural wall

4.4 Nonlinear Incremental Lateral Load Analysis

4.4.1 Modeling of Building

(1) A building structure may be idealized as a series of plane frames in a principal direction if the effect of torsion and transverse frames can be neglected.

(2) If the effect of torsion and transverse frames cannot be neglected, a building must be analyzed as a three dimensional structure. If the three–dimensional effect can be considered in a plane structural model, such plane structural model may be used.

(3) A structure shall, as a general rule, be analyzed including the super–structure, basement and foundation structure.

4.4.2 Lateral Load Distribution

The distribution of lateral loads shall be the same as the one assumed in the serviceability limit state design (Section 4.2.1); the distribution in the basement shall meet the requirements of Notification No. 96 of Bureau of Housing.

4.4.3 Lateral Loading Analysis

(1) Horizontal loads shall be assumed to act at the floor level of each floor.

(2) The analysis may be terminated when the maximum story drift angle of a story reaches the design proof drift angle R_{u2}.

4.4.4 Modeling of Structural Members

(1) A column and girder shall be represented by a line member considering the following deformations:

Column: Bending, shear and axial deformations,

Girder: Bending and shear deformations.

(2) A beam–column connection may be assumed to deform in shear, or to be rigid in a region specified in Commentary of Article 8.2 in "AIJ Standard for Structural Calculation of Reinforced Concrete Structures."

(3) Inelastic deformation of a column and girder may be assumed to concentrate at the member end, represented by rotation of a rigid–plastic rotational spring.

(4) Shear, flexural and axial deformations of a structural wall shall be included.

4.5 Stiffness and Strength of Structural Members

(1) Restoring Force Characteristic Model

Stiffness change at cracking and flexural yielding shall be considered in restoring characteristics of a member.

(2) Ultimate Resistance of Member

Ultimate resistance of a member shall be evaluated by using the material strength specified in Chapter 3.

CHAPTER 5 EARTHQUAKE RESISTANT DESIGN (ELASTIC ANALYSIS PROCEDURE)

5.1 Design Principles

5.1.1 Scope of Buildings

This chapter may be used for the design of buildings less than 31 m in height.

5.1.2 Serviceability and Ultimate Limit State Design

The earthquake resistant design of super–structure of a building shall be examined for serviceability and ultimate limit states. The serviceability limit state design examines the performance criteria, specified in Section 5.2, by a linearly elastic analysis. The ultimate limit state design examines the performance criteria, specified in Section 5.3, at the formation of a collapse mechanism.

5.2 Serviceability Limit State Design

5.2.1 Design Earthquake Loads

Design earthquake load shall be the same as the one specified in 4.2.1.

5.2.2 Performance Criteria

The super–structure of a building shall satisfy the following conditions under the design earthquake load;

(1) No flexural yielding shall develop in structural members, and

(2) Story drift angle at each story shall not exceed the following limiting values;

for $0.00 < b_s < 0.30$: 1/600 rad

for $0.30 < b_s < 0.70$: 1/800 rad

for $0.70 < b_s < 1.00$: 1/1000 rad

where, b_s: the ratio of base overturning moment resisted by structural walls under design earthquake load.

5.3 Linearly Elastic Analysis

5.3.1 Modeling of Building

A building structure may be idealized as specified in Section 4.4.1.

5.3.2 Modeling of Structural Members

(1) A column and beam shall be represented by a line member considering the following deformations:

Column: Bending, shear and axial deformations, Beam: Bending and shear deformations.

(2) A beam–column connection may be assumed to be rigid in a region. If a deep girder is connected at the connection, shear deformation shall be considered in the connection. The rigid zone at a column and girder end shall be determined as specified in Commentary of Article 8.2 of "AIJ Standard for Structural Calculation of Reinforced Concrete Structures."

(3) Shear, flexural and axial deformations of a structural wall shall be included in the model.

5.3.3 Stiffness of Structural Members

(1) The stiffness of a column, girder and structural wall shall, as a general rule, be linearly elastic.

(2) The effect of orthogonal elements shall be considered in the flexural stiffness of a member.

5.3.4 Stiffness Reduction of Structural Members

(1) In structural members, subjected to high local stresses in the linear analysis, the elastic stiffness may be reduced.

(2) Those members whose stiffness may be reduced shall be limited to short–span girders and girders connected to a structural wall.

5.3.5 Structural Walls with Opening

(1) In a structural wall with an opening, the stiffness shall be properly reduced.

(2) The allowable location of an opening in a structural wall shall be specified in Chapter 7.

5.4 Ultimate Limit State Design

Super–structure shall satisfy the conditions (1) and (2) listed below.

(1) Lateral Load Resisting Capacity

The lateral load resisting capacity of a structure shall be larger than the required lateral load resisting capacity. The required lateral load resisting capacity Quni of story i is defined by Eq. (5.4.1):

$$Q_{uni} = C_{uni}\,W_i \tag{5.1}$$

$$C_{uni} = Z\,R_t\,A_i\,C_{uns} \tag{5.2}$$

where, Cuni: Story shear coefficient for ultimate limit state design of story i, Wi: Sum of dead and live (corresponding to earthquake load) loads supported at story i, Cuns: standard base shear coefficient for ultimate limit state design, and shall be not less than the value specified below:

for $0.0 < bs < 0.3$, $Cuns = 0.30$

for $0.3 < bs < 0.7$, $Cuns = 0.35$
for $0.7 < bs < 1.0$, $Cuns = 0.40$

where, bs: the ratio of overturning moment resisted by structural walls at the level of first story floor to the total overturning moment evaluated at the formation of collapse mechanism.

(2) Collapse Mechanism

In order to assure the formation of a total collapse mechanism of the structure, the locations and members of planned yield hinges and also the locations and members not allowed to form yield hinges shall be designed for the action at the formation of collapse mechanism magnified by the amplification factor of design member action.

5.5 Ultimate Strength of Members

The ultimate strength of members shall be evaluated as specified in Section 4.5.2.

5.6 Amplification Factors of Design Member Action

5.6.1 Amplification Factors

The amplification factors of design member actions shall satisfy either (1) or (2) below.

(1) Members with Planned Yield Hinge

(a) The amplification factors for each design member action shall be taken from Table 5.6.1.

(b) Design bending moment shall be used for a region other than the yield hinge.

(c) Design bond stress shall be calculated using the amplified actions.

(2) Members without Planned Yield Hinge

(a) The amplification factors for each design member action shall be taken from Table 5.6.2.

(b) Ultimate flexural strength of columns without a planned yield hinge shall be evaluated using the specified yield strength of longitudinal reinforcement.

(c) Ultimate flexural strength of walls without a planned yield hinge shall be evaluated using the specified yield strength of longitudinal reinforcement.

(d) Shear strength of members shall be evaluated using the specified yield strength of transverse reinforcement.

(e) Design bond stress shall be calculated using the amplified actions.

Table 5.6.1: Amplification factors of design actions in member yielding at one or both ends

Member	Actions	Amplification Factors
(a) Girders	Bending	a_1
(b) Columns	Bending	$a_2 + 0.1$
	Shear	a_3
(c) Walls	Bending	a_4
	Shear	a_5

a_1 to a_5 are the same as ones defined in Table 4.3.2.

Table 5.6.2: Amplification factors of design actions in member without yielding

Member	Actions	Amplification Factors
(a) Columns	Bending	$a_2 + 0.1$
	Shear	a_3
(b) Walls	Bending	a_4
	Shear	a_5
(c) Beam–column Joint	Shear	a_6

a_2 to a_6 are the same as ones defined in Table 4.3.3.

5.6.2 Limitation of Column Axial Load

Column axial load N, calculated at the formation of collapse mechanism, shall stay within the region specified below:

$$(3/4)\ Nt < N < (2/3)\ Nu \qquad (5.3)$$

where, Nu = Ac Fc, Nt = Ag fy, Ac: column cross sectional area, Fc: specified concrete strength, Ag: gross sectional area of column longitudinal reinforcement, and fy: material strength of column longitudinal reinforcement.

5.6.3 Design of PCa Connection

Design actions at a PCa connection shall be evaluated by the existing forces calculated at the formation of collapse mechanism magnified by the amplification factors specified in Section

5.6.1. Design of PCa connections shall conform to the provisions of "PRESSS Design Manual for PCa Connections."

CHAPTER 6 DESIGN OF FOUNDATION AND BASEMENT

6.1 Method of Design

6.1.1 Design Principle

Foundation and basement structures shall be designed to satisfy the requirement of Chapter 1 and the performance criteria specified in Section 2.1 for the loading under gravity, high winds, earthquakes and snow falls.

Foundation and basement structures shall be designed with care if special design conditions, such as ground settlement and liquefaction, need to be considered.

6.1.2 Design Actions

Design stresses for foundation and basement structures shall be determined by (a) and (b) below;

(1) Design stresses for basement structures shall include the effect of soil and hydraulic pressure in addition to the effect of long-term gravity load and short-term snow, wind pressure and earthquake loads. Stresses caused by uneven settlement and lifting or deflection of the soil and piles shall be considered if appropriate.

(2) Design stresses for foundation structures shall include the effect of surrounding soil in addition to the effect of long-term gravity load and short-term snow load and wind pressure and earthquake loads. Stresses caused by uneven settlement and lifting or deflection of the soil and piles shall be considered if applicable.

6.1.3 Design of Structural Members

(1) Design of structural members in the foundation and basement shall satisfy (a) to (d) below:

(a) Design stresses due to the long-term gravity load and short-term snow load and wind pressure shall not exceed allowable stresses of materials specified in Section 3.3.

(b) At an earthquake load level for the ultimate limit state of the super-structure, no flexural yielding nor shear failure shall, as a general rule, occur in structural members in the foundation and basement. The location and members, where flexural yielding is permitted at the ultimate limit state, shall not yield at the serviceability limit state.

(c) Axial force in piles at the long-term gravity loading shall not exceed the allowable vertical bearing capacity of the soil for the long-term loading, and stress in the pile shall not exceed the allowable stress of the pile for the long-term loading. Axial force in piles at an earthquake load level for the serviceability limit state of the super-structure and the short-term snow and wind loading shall not exceed the allowable bearing capacity of soil for the short-term loading, and stresses in piles shall not exceed the allowable stresses of piles for the short-term loading.

At an earthquake load level for the ultimate limit state of the super-structure, axial force in piles shall not exceed the ultimate bearing capacity of soil, and the piles shall not, as a general rule, develop flexural yielding nor shear failure.

(d) Ground contact pressure of direct foundation under the long-term gravity loading shall not exceed the allowable bearing pressure of the soil for the long-term loading; ground contact pressure at an earthquake load level for the serviceability limit state of the super-structure and under the short-term snow and wind pressure shall not exceed the allowable bearing stress of soil for the short-term loading.

At an earthquake load level for the ultimate limit state of the super-structure, the bearing pressure shall not exceed the bearing capacity of soil.

(2) The reduction of vertical and horizontal load resistance in sandy soil and clay soil below the ground water level shall be properly considered under earthquake loading.

6.1.4 Examination of Foundation Uplifting

Foundation shall not be uplifted under the earthquake load level for the serviceability limit state of the super-structure.

6.2 Design of Direct Foundation

6.2.1 Design for Ground Contact Pressure

(a) Ground contact pressure due to loads for the long-term loading, specified in Section 2.2.2, shall not exceed the allowable bearing stress of soil for the long-term loading.

(b) Ground contact pressure due to loads for the short-term loading by snow load and wind pressure, specified in Sections 2.2.4 and 2.2.5, shall not exceed the allowable bearing stress of soil for the short-term loading.

(c) Ground contact pressure at an earthquake load level for the serviceability limit state of the super-structure shall not exceed the allowable bearing stress of soil for the short-term loading.

(d) Ground contact pressure at an earthquake load level for the ultimate limit state of the super-structure shall not exceed the bearing capacity of soil. Ground contact pressure caused by maximum earthquake loading in any direction shall not exceed the bearing capacity of soil.

6.2.2 Allowable Bearing Stresses and Ultimate Bearing Strength

(1) The allowable bearing stress for direct foundation shall be determined not to exceed the allowable bearing capacity of soil, and not to cause uneven settlement which affects the serviceability of a structure.

(2) The ultimate bearing stress for direct foundation shall be determined not to cause excessive tilting of a structure due to settlement, not to cause yielding in members other than the planned members, and not to cause brittle failure, such as shear and bond splitting failure, of foundation and principal structural members.

6.2.3 Design for Horizontal Loading

If horizontal force acts on the lower face of the direct foundation, the safety against sliding shall be examined.

6.2.4 Soil beneath Foundation Slab

Foundation slab of direct foundation shall rest on stable soil which shall not result in volume change or liquefaction under gravity and earthquake loading.

6.3 Design of Pile Foundation

6.3.1 Principles

(a) Vertical load capacity of pile foundation shall, as a general rule, be axial load bearing capacity of the pile itself.

(b) Design force for pile foundation shall be horizontal and vertical actions transmitted from the floor immediately above the foundation and the load acting on the foundation. Impact, cyclic, eccentric, and inclined loads shall be included if applicable.

(c) If the ground may become unstable due to ground settlement, lateral movement and liquefaction during an earthquake, the effect shall be considered in design.

6.3.2 Design for Vertical Loads

(a) Vertical force on a pile due to the long-term loads, specified in Section 2.2.2, shall not exceed the allowable bearing force of the pile for the long-term loading. If pull-out force acts on a pile under the long-term loading, the force shall not exceed the allowable pull-out force of the pile for the long-term loading.

(b) Vertical force on a pile due to loads for the short-term loading by the snow and wind pressure, specified in Sections 2.2.4 and 2.2.5, shall not exceed the allowable bearing force of the pile for the short-term loading. If pull-out force acts on a pile under the short-term loading, the force shall not exceed the allowable pull-out force of the pile for the short-term loading.

(c) Vertical force of a pile at an earthquake load level for the serviceability limit state of the super-structure shall not exceed the allowable bearing force of the pile for the short-term loading. If pull-out force acts on a pile, the force shall not exceed the allowable pull-out force of the pile for the short-term loading.

(d) Vertical force on a pile at an earthquake load level for the ultimate limit state of the super-structure shall not exceed the bearing capacity of the pile. If pull-out force acts on a pile, the force shall not, as a general rule, exceed the ultimate pull-out strength of the pile. The vertical force caused by maximum earthquake loading in any direction shall not exceed the bearing capacity of the pile.

6.3.3 Design for Horizontal Loads

(a) If a horizontal force acts on a pile under the long-term loading, the stress developed in the pile shall not exceed the allowable stresses of materials for the long-term loading.

(b) Stress in a pile under the short-term loading by snow and wind pressure as well as under earthquake loads for the serviceability limit state of the super-structure shall not exceed the allowable stresses of materials for the short-term loading.

(c) At an earthquake load for the ultimate limit state of the super-structure, a pile shall, as a general rule, be provided with a required horizontal strength against shear failure in the pile and at the connection to the pile cap. Excessive horizontal deflection of a pile shall not occur. If flexural yielding is permitted in a pile under an earthquake loading level at the ultimate limit state of the super–structure, required horizontal resistance shall be maintained in the pile, and shear failure shall not occur in the pile and at the connection to the pile cap.

6.3.4 Connection of Piles

(1) Connection of a pile to a pile cap and to a foundation girder shall be designed by the same criteria as the pile foundation.

(2) Splicing of a pile shall be provided with resistance sufficient to transmit actions developed at the locations.

6.4 Design of Foundation Slab and Girder

6.4.1 Principles

(1) Design of a foundation slab for direct foundation shall satisfy (a) to (e) below:

(a) Stresses in a foundation slab due to ground bearing pressure under the long–term loading shall not exceed the allowable stresses of materials, specified in Section 3.3, for the long–term loading.

(b) Stresses in a foundation slab due to ground bearing pressure under the short–term loading by the snow load and wind pressure shall not exceed the allowable stresses of materials, specified in Section 3.3, for the short–term loading.

(c) At an earthquake load level for the ultimate limit state of the super–structure, a foundation slab shall not yield in flexure nor fail in a brittle manner, such as in shear, due to ground bearing pressure.

(d) A foundation slab shall not yield in flexure nor fail in a brittle manner, such as in shear, due to ground bearing pressure caused by maximum earthquake loading in any direction.

(e) Stresses developed in a foundation slab, described in (a) to (d) above, shall be transmitted to foundation girders.

(2) Design of a foundation slab for pile foundation and foundation slab shall satisfy (a) to (e) below:

(a) Stresses in a foundation slab due to the action in pile under the long–term loading shall not exceed the allowable stresses of materials, specified in Section 3.3, for the long–term loading.

(b) Stresses in a foundation slab due to the action in pile under the short–term loading by the snow load and wind pressure shall not exceed the allowable stresses of materials, specified in Section 3.3, for the short–term loading.

(c) At an earthquake load level for the ultimate limit state of the super–structure, a foundation slab shall not yield in flexure not fail in a brittle manner, such as in shear and punching shear, due to the action in pile.

(d) A foundation slab shall not yield in flexure not fail in a brittle manner, such as in shear and punching shear, due to the action in pile caused by maximum earthquake loading in any direction.

(e) Stresses developed in a foundation slab, described in (a) to (d) above, shall be transmitted to foundation girders.

(3) If PCa members are to be used in foundation slab, the PCa members shall be ensured to develop structural performance specified in Section 2.1.2. The action at the PCa connection shall be estimated under different loadings, and PCa connections shall be designed in accordance with "PRESSS Design Manual for PCa Connection."

6.4.2 Design of Foundation Girder

(1) Design stresses for a foundation girder shall be calculated for stresses from ground bearing pressure in direct foundation and actions in pile in pile foundation, in addition to stresses transmitted from connecting columns and structural walls. Stresses due to soil and hydraulic pressure and due to out-of-plane actions by piles shall be considered.

(2) Design of a foundation girder shall satisfy (a) to (d) below:

(a) Stresses in a foundation girder under the long-term loading shall not exceed the allowable stresses of materials, specified in Section 3.3, for the long-term loading.

(b) Stresses in a foundation girder under the short-term loading by the snow load and wind pressure shall not exceed the allowable stresses of materials, specified in Section 3.3, for the short-term loading.

(c) At an earthquake load level for the ultimate limit state of the super-structure, a foundation girder shall not, as a general rule, yield in flexure nor fail in shear and bond splitting. If a foundation girder is permitted to yield at an earthquake load level for the ultimate limit state of the super-structure, the girder shall not yield under the serviceability limit state.

(3) If PCa members are to be used in a foundation girder, the PCa members shall be ensured to develop structural performance specified in Section 2.1.2. The action at the PCa connection shall be estimated under different loadings, and PCa members shall be designed in accordance with "PRESSS Design Manual for PCa Connection."

6.4.3 Connection of Foundation Slab with Foundation Girder

Connection between foundation slab and foundation girders shall be provided with sufficient rigidity and strength to transfer the action developed in the foundation slab to the foundation girder.

6.5 Design of Members in Basement

(1) Design for structural members in basement shall satisfy (a) to (d) below:

(a) Stresses in members in basement due to the long-term loading shall not exceed the allowable stresses of materials, specified in Section 3.3, for the long-term loading.

(b) Stresses in members in basement due to the short-term loading by the snow load and wind pressure shall not exceed the allowable stresses of materials, specified in Section 3.3, for the short-term loading.

(c) At an earthquake load level for the ultimate limit state of the super–structure, structural members shall not, as a general rule, yield in flexure nor fail in shear and bond splitting modes.

(d) Axial force in a column under earthquake loading in any direction shall be less than 2/3 Nu (Nu: compressive strength of column), and greater than 3/4 Nt (Nt: tensile strength of column).

(2) If a PCa member is to be used in a part of basement to contact the ground, the PCa member shall satisfy (1) above, and also shall be ensured to develop structural performance specified in Section 2.1.2. PCa connections shall be designed in accordance with "PRESSS Design Manual for PCa Connection."

CHAPTER 7 STRUCTURAL REQUIREMENTS

7.1 General Requirements

Nominal bar size, spacing, clearance, cover depth, standard bend of reinforcement, if not specified in this guidelines, shall conform to "Building Standard Law Enforcement Order," "Architectural Institute of Japan Standard for Structural Calculation of Reinforced Concrete Structures, Japan Architectural Standard Specification (JASS) and its Commentary on Reinforced Concrete Work (JASS-5)," "Reinforcement Arrangement Guidelines for Reinforced Concrete Structures and its Commentary," and "PRESSS Guidelines for Construction and Quality Control of PCa Construction."

7.2 Columns

(1) Column Dimensions

The shorter dimension of a column section shall be not less than 40 cm. A ratio of section dimensions of long side to short side shall be not more than 2.

(2) Longitudinal Reinforcement

(a) Longitudinal reinforcement shall be deformed bars of size equal to or larger than D19.

(b) Gross reinforcement ratio of longitudinal reinforcement shall be not less than 0.008.

(3) Lateral Reinforcement

Lateral reinforcement shall be deformed bars of size equal to or larger than D10. Lateral reinforcement shall be arranged to effectively confine the longitudinal reinforcement and concrete. The spacing of lateral reinforcement shall satisfy the values specified in Table 7.2.1.

Table 7.1: Minimum Spacing of Column Lateral Reinforcement
(Unit: mm)

Special yield hinge	Yield hinge	Non-yield hinge
Spacing not more than yield hinge region Use sub-ties All long. bars be supported[3)]	(1) D / 5 (2) 150 (3) 6 d_b Use sub-ties[1)] Intermediate long. bars be supported[2)]	(1) D / 3 (2) 200 (3) 8 d_b

where, d_b: size of longitudinal reinforcement in mm.

1) Lateral reinforcement placed on intermediate longitudinal reinforcement.

2) Intermediate longitudinal reinforcement, placed more than 300 mm apart, shall be laterally supported by a corner of closed shape lateral reinforcement or 135 degree bend.

3) All longitudinal reinforcement, as a general rule, shall be laterally supported by a corner of closed shape lateral reinforcement or 135-degree bend. However, longitudinal reinforcement, within 200 mm between the two adjacent supported longitudinal reinforcement, may not be supported.

(4) Yield Hinge Region

A region, where flexural yielding may take place at a yield hinge, shall be equal to 1.5 times column depth from the orthogonal beam face.

(5) Special Yield Hinge Region

A special yield hinge region is defined as a yield hinge region where design axial force N_c at the design proof deformation specified in Chapter 4 or at the formation of a collapse mechanism specified in Chapter 5 fall in a region of Eq.(7.1.1).

$$1/3\ A_c F_c < N_c \qquad (7.1.1)$$

where, Ac: column sectional area, and Fc: specified concrete compressive strength.

(6) End of Lateral Reinforcement in Special Yield Hinge Region

End of lateral reinforcement within a special yield hinge region shall conform to (a) to (c) below where d_b: bar diameter of lateral reinforcement;

(a) The end of lateral reinforcement other than closed shape welded lateral reinforcement and spiral reinforcement shall be anchored with 135-degree hook, and with extension of more than 8 d_b,

(b) The end of lateral reinforcement other than of closed shape shall be bent more than 135 degrees. The extension shall be more than 8 d_b for 135 degree bend, and more than 4 d_b for 180-degree bend.

(c) The end of spiral reinforcement shall be anchored into the confined core concrete with 1-1/2 extra turns. The end of spiral reinforcement shall be provided with hooks of 135-degree bend with extension of more than 8 d_b or hooks of 90-degree bend with extension of more than 14 d_b.

7.3 Beams and Girders

This section specifies requirements for beams and girders other than sub-beams.

(1) Sectional Shape

Width of a beam shall be not less than 25 cm. Width of a beam in a yield hinge region shall be not less than one-quarter of beam depth.

(2) Longitudinal Reinforcement

(a) Longitudinal reinforcement shall be deformed bars of size equal to or larger than D16.

(b) Area ratio of compressive to tensile reinforcement shall be not less than 0.5.

(c) Tensile reinforcing bars shall, as a general rule, be placed in not more than two layers in the section.

(3) Anchorage of Second–Layer Longitudinal Reinforcement

The cut–off location of beam longitudinal reinforcement from the critical section shall be determined considering the anchorage length sufficient for the stress in the longitudinal reinforcement to be safely transferred by bond.

(4) Lateral Reinforcement in Beam

Lateral reinforcement in beam shall be deformed bars of size equal to or greater than D10. Lateral reinforcement shall be arranged to effectively confine the longitudinal reinforcement and concrete. The spacing of lateral reinforcement shall satisfy the values specified in Table 7.3.1.

Table 7.3.1: Minimum Spacing of Beam Lateral Reinforcement
(Unit: mm)

Yield hinge region	Non–yield hinge region
(1) D / 3	(1) D / 3
(2) 200	(2) 300
(3) 8 d_b	

where, d_b: size of longitudinal reinforcement in mm, D: depth.

(5) Yield Hinge Region

A yield hinge region is defined as a region where flexural yielding takes place, and shall be equal to 1.5 times beam depth from the orthogonal column face.

(6) Small Openings

(a) The diameter of an opening shall be not more than one–third of the beam depth.

(b) Center–to–center spacing of two adjacent openings shall be more than three times the diameter of the larger opening.

(c) The distance from the column face to the edge of an opening shall be, as a general rule, more than the beam depth. However, the distance requirement may not be satisfied if the safety is proven by a special study, such as experiment.

7.4 Structural Walls

(1) Sectional Shape

(a) A structural wall, as a general rule, shall be provided with boundary columns at both edges.

(b) Thickness of a structural wall shall be not less than 150 mm, and not less than 0.05 times clear story height.

(2) Reinforcement

(a) Reinforcement shall be deformed bars of size not less than D10.

(b) Reinforcement shall be placed in double layers in a yield hinge region.

(3) Opening

An opening shall not be placed, as a general rule, in a yield hinge region. If an opening is to be placed in a non-yield hinge region, the opening shall be placed near the center portion of the wall span. The size of the opening shall be selected not to alter the structural characteristics of the structural wall.

(4) Minimum Reinforcement

Lateral reinforcement ratio shall be not less than 0.0025. The ratio shall be not less than 0.003 in a yield hinge region.

(5) Lateral Reinforcement

The spacing of vertical reinforcement shall be not more than 300 mm, and the spacing of lateral reinforcement shall satisfy the values specified in Table 7.4.1.

Table 7.4.1: Minimum Spacing of Wall Lateral Reinforcement
(Unit: mm)

Yield hinge region	Non-yield hinge region
200	300

(6) Sub-Ties

Sub-ties using deformed bars of size not less than D10 shall be placed at 300 mm spacing over 1/5 clear span and over the entire height within a yield hinge region.

(7) Anchorage of Wall Reinforcement

All wall reinforcement, as a general rule, shall be anchored either in boundary columns or in boundary beams.

(8) Yield Hinge Region

A yield hinge region is defined as a region where flexural yielding takes placed and shall be equal to the larger of 1/6 times the wall height or the horizontal wall dimension from the first story wall base. However, the height of a yield hinge region may not be taken higher than the bottom face of the third floor beam.

(9) Special Yield Hinge Region

A special yield hinge region is defined as a yield hinge region where axial force N_w at the design proof deformation in Chapter 4 or at the formation of the collapse mechanism in Chapter 5 falls in a region specified in Eq.(7.4.1).

$$2/3\, A_{core}\, F_c - A_{ws}\, f_{ws} < N_w < A_{core}\, F_c - A_{ws}\, f_{wy} \tag{7.4.1}$$

where, A_{core}: core area of a compressive side boundary column, A_{ws}: total area of longitudinal reinforcement in wall panel, F_c: specified compressive strength of concrete, f_{ws}: material strength of longitudinal reinforcement in wall panel.

(10) Lateral Reinforcement in Special Yield Hinge Region

Lateral reinforcement of a boundary column within a special yield hinge region shall satisfy the requirements in Section 7.2 (3) for spacing, and Section 7.2 (6) for detailing.

7.5 Beam-Column Connections

(1) Lateral Reinforcement

Lateral reinforcement ratio in a beam-column connection shall be not less than 0.003. Spacing of lateral reinforcement shall satisfy the values specified in Table 7.5.1.

Table 7.5.1: Minimum Spacing of Lateral Reinforcement in Beam-column Connection (unit: mm)

Deformed bar D10[1)]	150
Deformed bar beyond D10[2)]	200 and 8 d_b

where, d_b: nominal diameter of longitudinal reinforcement.

Note 1): including high strength bars of nominal diameter of not less than 6 mm and less than 11 mm;

2): including high strength bars of nominal diameter of not less than 11 mm.

(2) Anchorage of Beam and Column Reinforcement

(a) Anchorage Method

Beam longitudinal reinforcement shall, as a general rule, pass through or anchored with 90-degree bend in the column core of a beam-column connection. Beam longitudinal reinforcement in a yield hinge region shall be placed inside the column longitudinal reinforcement. Column longitudinal reinforcement, except at the base of the base story and at the top of the highest story, shall pass through a beam–column joint. The anchorage length shall be measured from the column face for beam longitudinal reinforcement, and from the beam face for column longitudinal reinforcement.

(b) Anchorage with Bend

If beam reinforcement is to be anchored with bend, horizontal lead length of longitudinal reinforcement shall be not less than 16 times the nominal bar diameter, and shall be sufficient to develop design anchorage force. The angle of bend shall be 90 degrees, and the bend shall start outside the column center line.

The extension shall be not less than 10 times nominal bar diameter placed within the beam–column joint.

(c) Bars Passing through Connection

If yield hinges are formed at the both faces of a beam–column connection at the Design Proof Deformation (at the formation of a collapse mechanism in Chapter 5), and if beam or column reinforcement passes through the connection, the column width (beam depth) shall, as a general rule, be not less than 25 times the nominal bar diameter.

7.6 Slabs and Sub–Beams

(1) Thickness of floor and roof slabs shall be not less than 130 mm.

(2) Slab Reinforcement

Reinforcing bars in slab shall be deformed bars of size not less than D10 or slab reinforcement shall be welded wire mesh of nominal diameter not less than 6 mm.

(3) Slab reinforcement ratio in both directions shall, as a general rule, be not less than 0.002 of concrete cross section.

(4) Full PCa slabs and half PCa slabs shall, as a general rule, satisfy the performance criteria on vibration characteristics and deformation of RC slabs.

(5) Thickness of cast–in–situ concrete in half PCa slabs shall be not less than 70 mm.

(6) Design of a connection between PCa slabs and a connection between a PCa sub–beam and its support shall conform to "PRESSS Design Manual for PCa Connection."

(7) PCa slab shall, as a general rule, be supported along an edge over not more than 30 mm.

(8) Minimum reinforcement ratio in a sub–beam shall conform to the requirements of "AIJ Standard for Structural Calculation of Reinforced Concrete Structures."

7.7 Foundation Girders and Foundation Slabs

(1) Structural requirements for foundation girders shall conform to the requirements of Section 6.4 and (a) to (c) below:

(a) Base of bottom story columns and structural walls, as a general rule, shall be connected effectively by foundation girders.

(b) Width of a foundation girder shall be not smaller than the thickness of a connected structural wall, and shall be, as a general rule, not smaller than the depth of a connected column.

(c) A foundation girder shall be, as a general rule, cast-in-situ reinforced concrete construction. If requirements in Section 6.4.2 (c) are satisfied, PCa members may be used for the foundation girder.

(2) Reinforcement in foundation girders shall be determined by structural calculation and shall satisfy (a) to (d) below:

(a) Longitudinal reinforcement shall be placed at the top and bottom of a section.

(b) Longitudinal reinforcement shall be, as a genral rule, placed not more than two layers at the top and at the bottom of a section.

(c) Spacing of longitudinal reinforcement in the vertical direction shall be, as a genral rule, not more than 300 mm. The amount of longitudinal reinforcement in a foundation girder supporting a structural wall shall be provided sufficient to resist the tensile strength of horizontal wall reinforcement.

(d) Spacing of lateral reinforcement shall, as a genral rule, be not more than 300 mm. The amount of lateral reinforcement in a foundation girder supporting a structural wall shall be provided sufficient to resist the tensile strength of wall vertical reinforcement.

(3) Reinforcement and structure of a foundation slab shall conform to the requirements of Chapter 6.4 and (a) to (c) below:

(a) Size and depth of a foundation slab shall be determined taking into consideration soil bearing pressure, pile size and number, and reaction from piles.

(b) Reinforcement in a foundation slab shall be determined by design calculation, and shall satisfy the minimum reinforcement specified in "AIJ Standard for Structural Calculation of Reinforced Concrete Structures."

(c) A foundation slab shall be, as a general rule, cast-in-situ reinforced concrete construction. If the requirements in Section 6.4.1 (3) are satisfied, PCa members may be used for the foundation slab.

CHAPTER 8 NON-STRUCTURAL ELEMENTS

8.1 Method of Design

(1) If a reinforced concrete or similar non-structural wall is to be installed in a structure, structural joints shall, as a general rule, be place along appropriate edges to separate the non-structural wall from structural members to ensure the structural performance at the ultimate limit state specified in Chapters 4 and 5.

(2) Non-structural elements shall be provided with strength sufficient to resist inertia force developed by an earthquake motion, and shall follow the structural deformation.

8.2 Connection of Non-Structural Elements

(1) Structural joints for non-structural walls shall, as a general rule, be either of a complete separate type or of a shear failure type.

(2) Stresses developed in a non-structural element and its connection by the inertia force of an earthquake motion shall be less than the allowable stress of the material for the short-term loading.

(3) A non-structural element and its connection shall not fail nor fall by the forced deformation expected at the ultimate limit state design.

NOTA BIOGRAFICA DE SHUNSUKE OTANI

El profesor Otani, graduado de la Universidad de Tokio en 1966, obtuvo sus grados de maestría y doctorado en la Universidad de Illinois en Urbana–Champaign por su trabajo desarrollado sobre el análisis de la respuesta no lineal ante cargas sísmicas de edificios de concreto reforzado.

Después de una corta estancia como profesor de la Universidad de Illinois, fue docente en el Departamento de Ingeniería Civil de la Universidad de Toronto entre 1975 y 1979. En 1979 fue nombrado como profesor asociado en el Departamento de Arquitectura de la Universidad de Tokio, y en 1993 fue ascendido a profesor titular de estructuras.

Participó en las investigaciones de los daños provocados por el sismo de México de 1985, en su calidad de secretario del equipo de investigación enviado por el Instituto de Arquitectura del Japón.

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