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PUBLICADO POR EL CENTRO NACIONAL DE
PREVENCION DE DESASTRES DE LA
SECRETARIA DE GOBERNACION

Distribución en México Coordinacion de Enlace Nacional
Distribución en el Exterior Coordinacion de Asuntos Internacionales

Agosto - 1994, No 10

Sistema Nacional de Protección Civil

SISTEMA NACIONAL DE PROTECCION CIVIL

CENTRO NACIONAL DE PREVENCION DE DESASTRES

PRACTICA DE DISEÑO Y CONSTRUCCION EN EL JAPON

JAPANESE PRESS DESIGN GUIDELINES FOR REINFORCED CONCRETE BUILDINGS

Shunsuke Otani

COORDINACION DE INVESTIGACION

AREA DE ENSAYES SISMICOS

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,

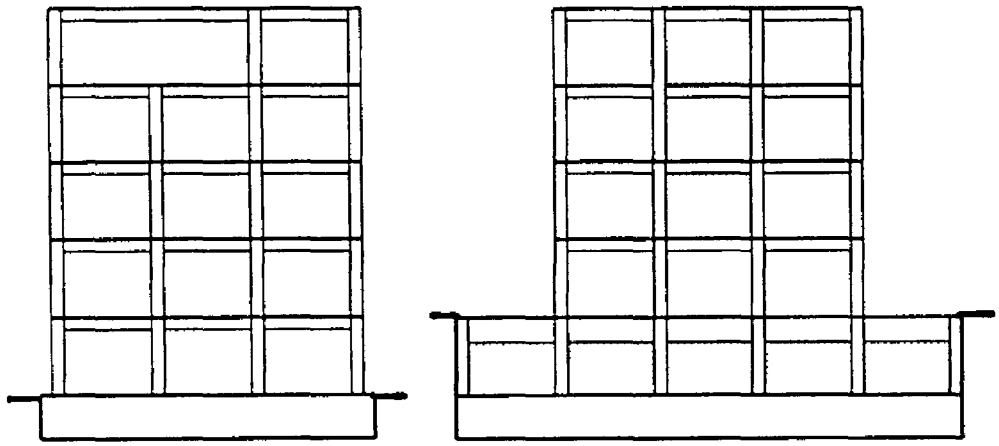
(a) Standard for the Evaluation of PCa Connection Performance,

(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.

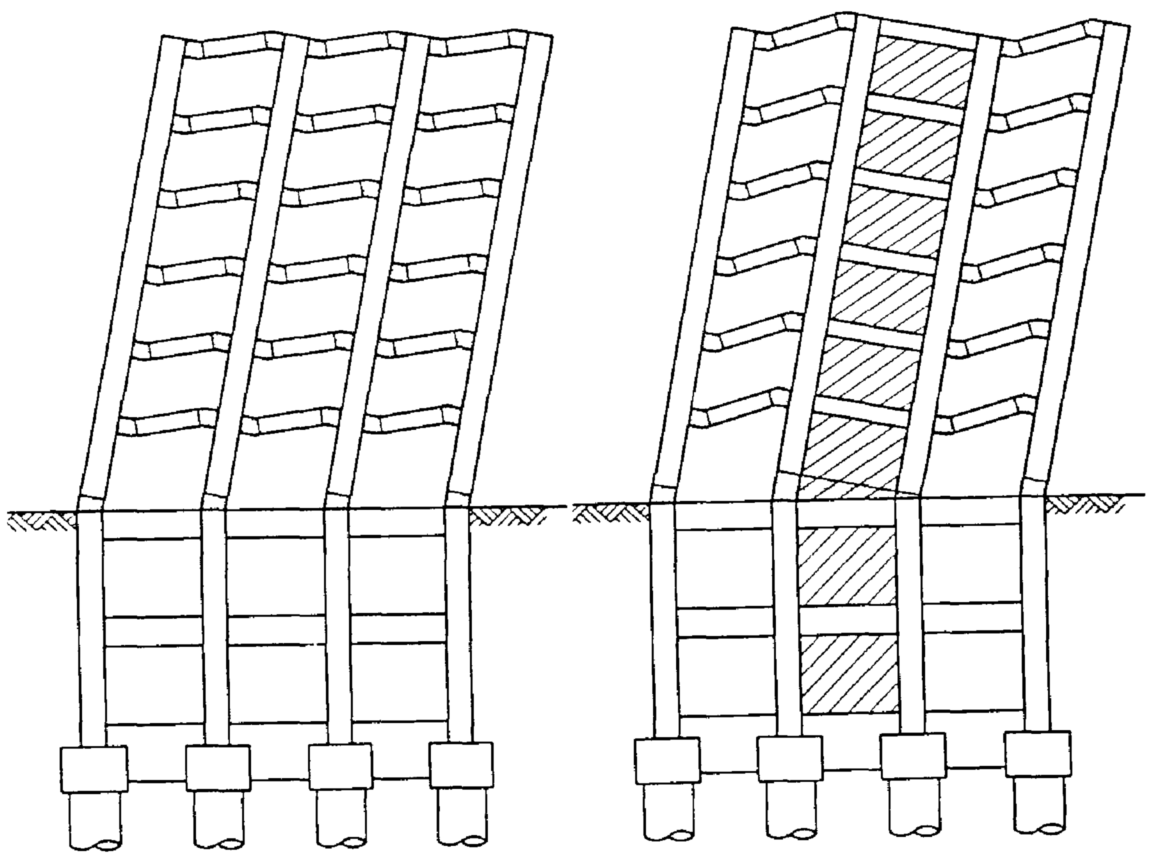
Ultimate Limit State -- A structure must be usable, even with an extensive repair work, after a high intensity earthquake motion, which may occur once in the lifetime of the structure.

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

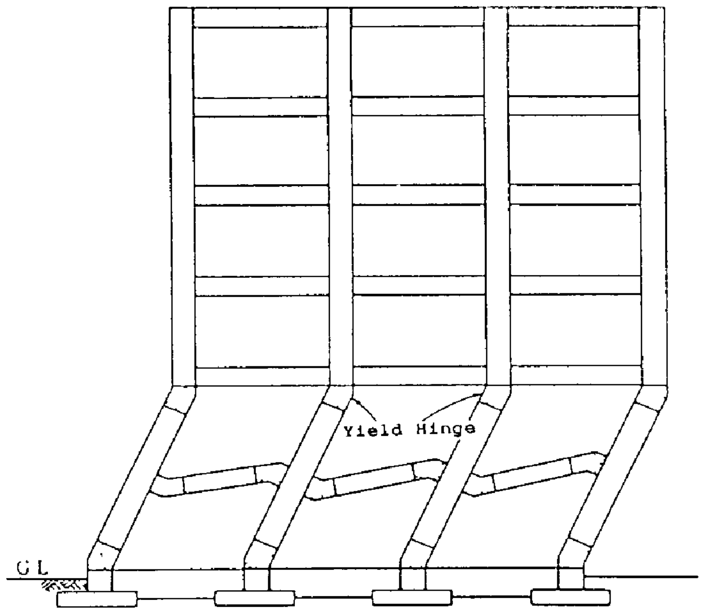


Fig. 3: Partial Yield Mechanism

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.

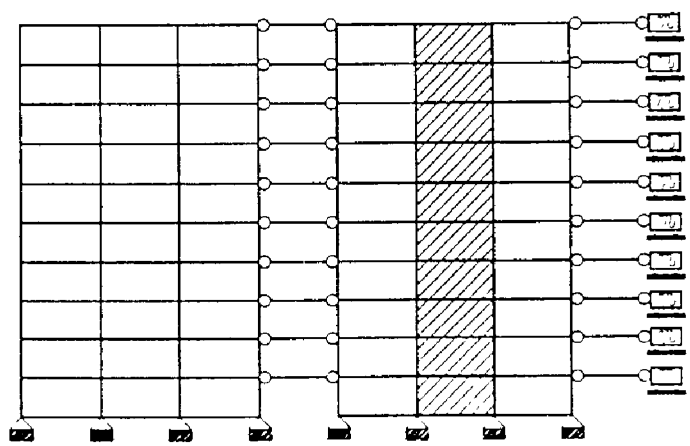


Fig 4 Member Modeling of Plane Frames

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.

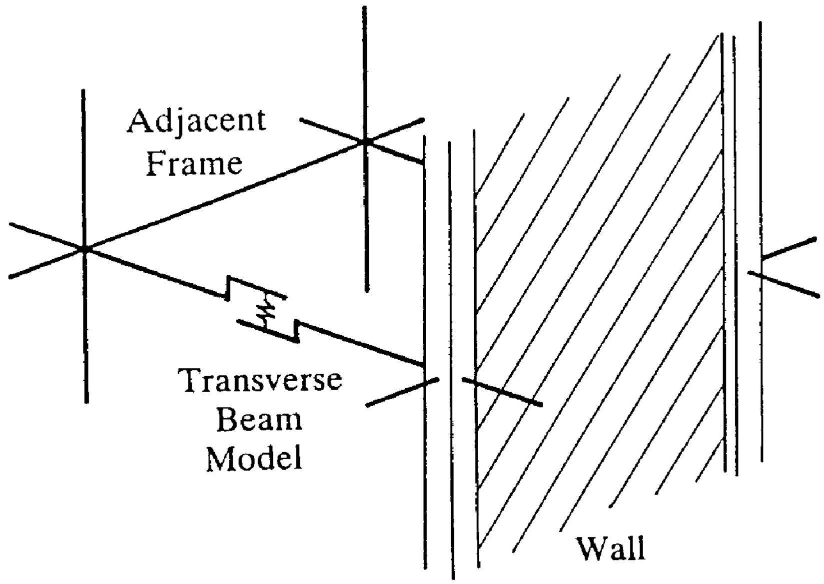
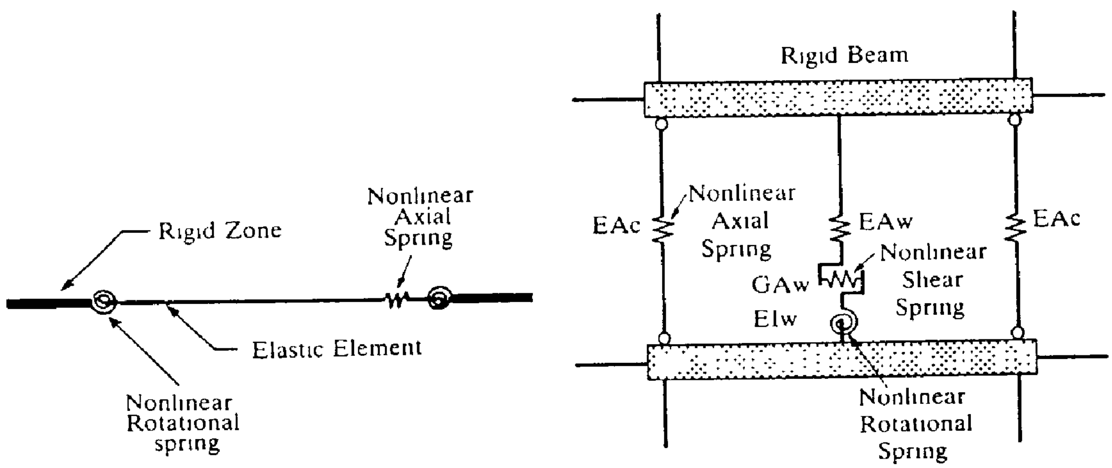


Fig. 5: Modeling of Structural Wall with Connecting Girders



(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.

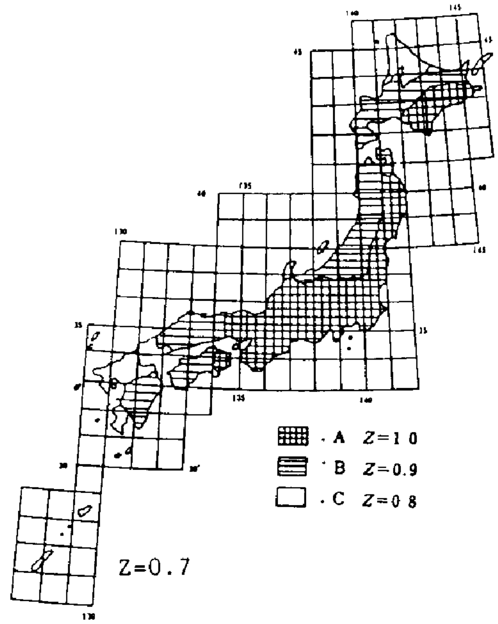


Fig. 7: Seismic Zone Factor

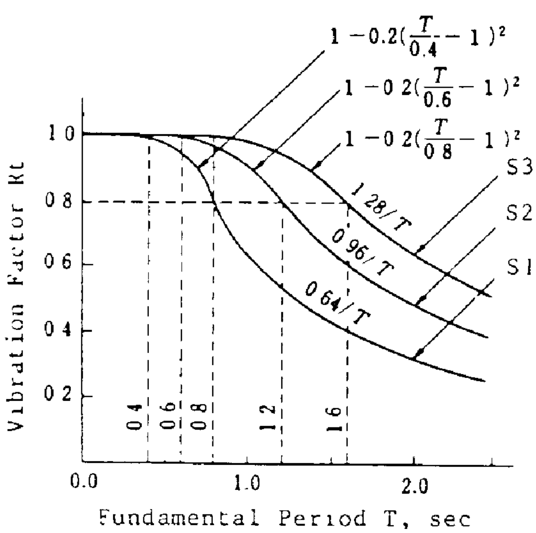


Fig. 8. Vibration Characteristic Factor

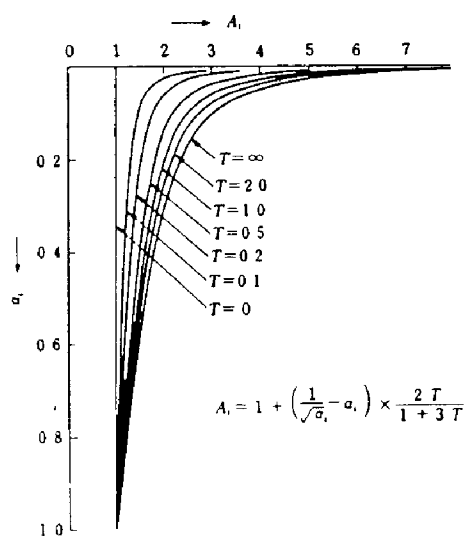


Fig. 9: Story Shear Distribution Factor

The coefficient A_i is given by the following expression:

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

where,

$$\alpha_i = W_i / W_1 \tag{8}$$

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.

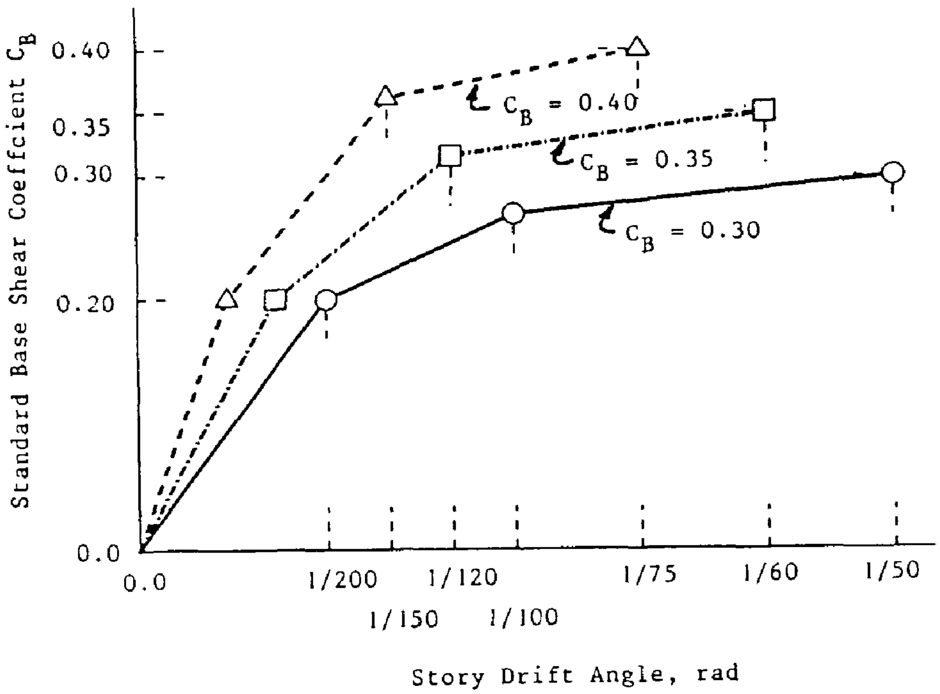


Fig. 10: Performance Criteria for Frame–Wall Structures

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.

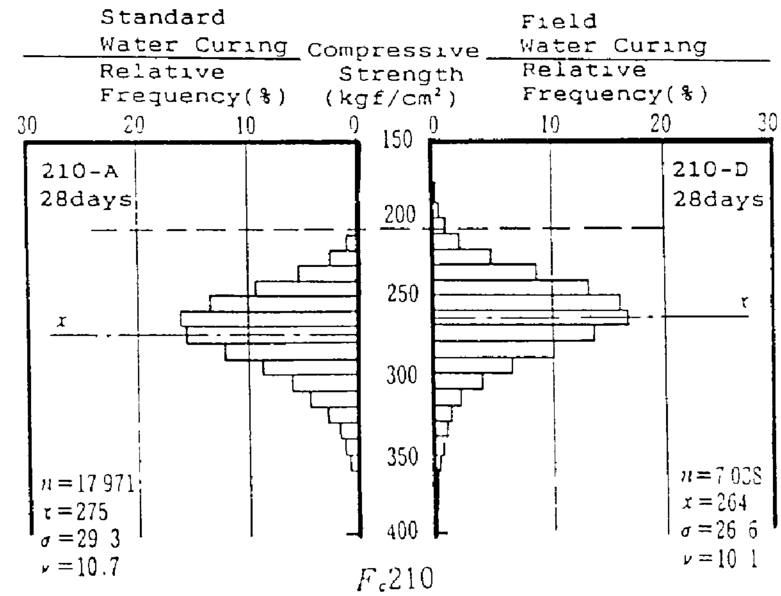


Fig. 11: Statistical Distribution of Concrete Strength

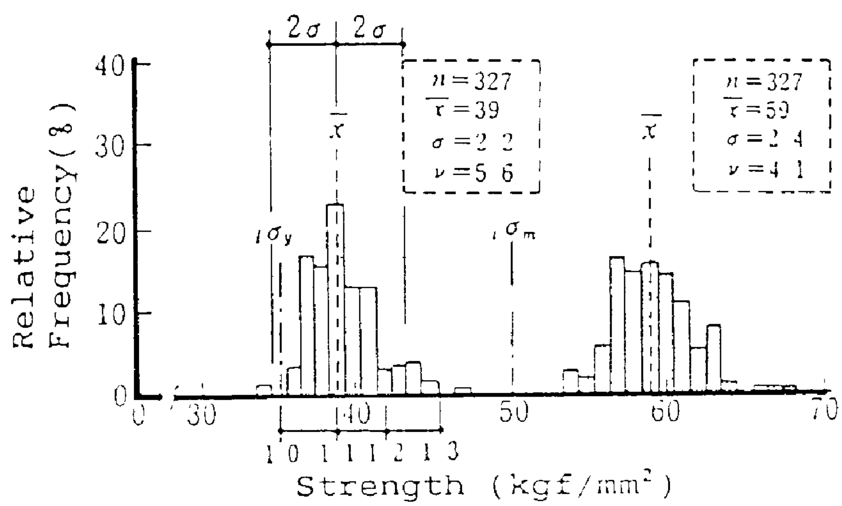


Fig. 12: Statistical Distribution of Yield Strength in Reinforcing Steel

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).

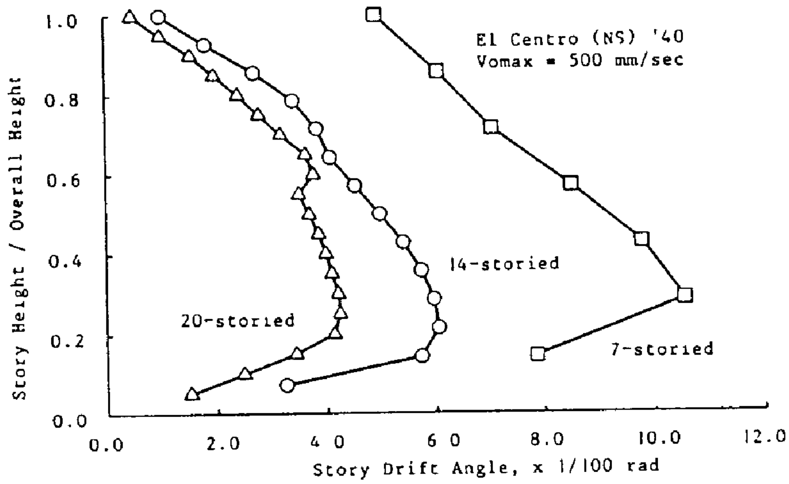
Although a shear distribution among columns of a story may vary with time during an earthquake, the dynamic amplification factor for story shears is used for column shears. Column end moments fluctuate more during an earthquake attributable to the shift of an inflection point. Although higher mode oscillation 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.

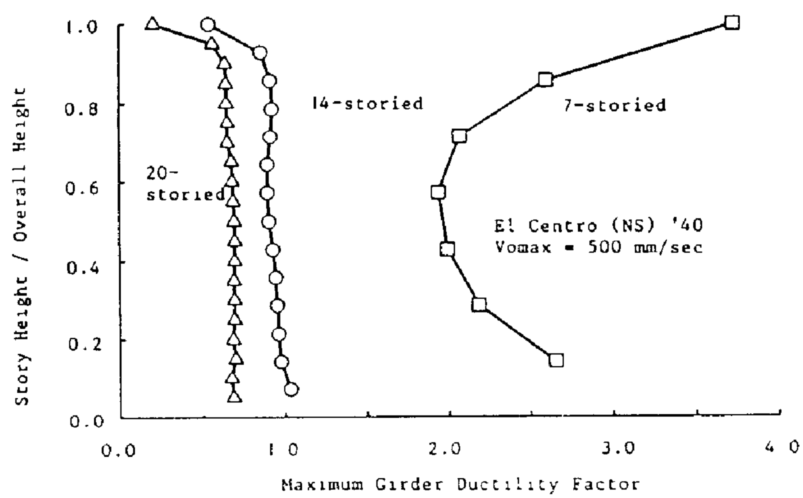
A series of simple structures of 5-story (15 m tall) to 20-story (60 m tall) buildings were analyzed under design earthquake motions, and maximum column actions were calculated Dynamic amplification factors were determined as ratios of maximum column actions (basic member forces) to the corresponding 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.



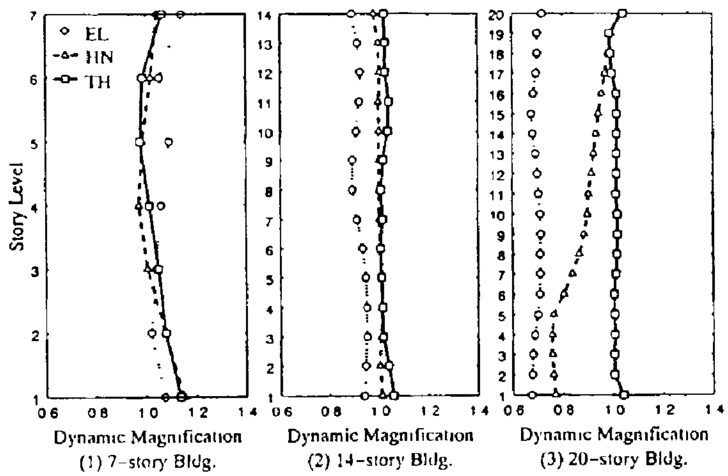
(a) Maximum Story Drift Angle



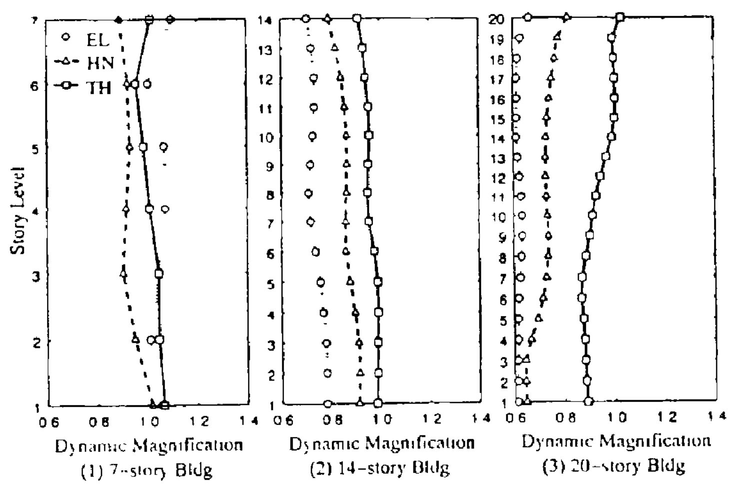
(b) Maximum Girder Ductility Demand

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.

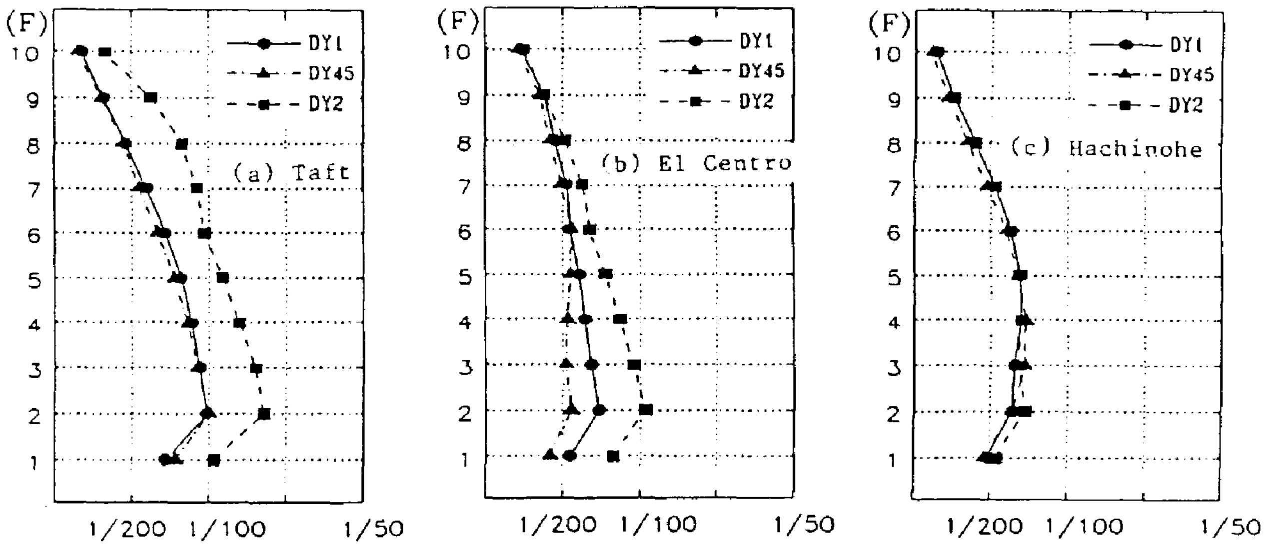


Fig. 15: Maximum Response under Uni-directional Earthquake Motion

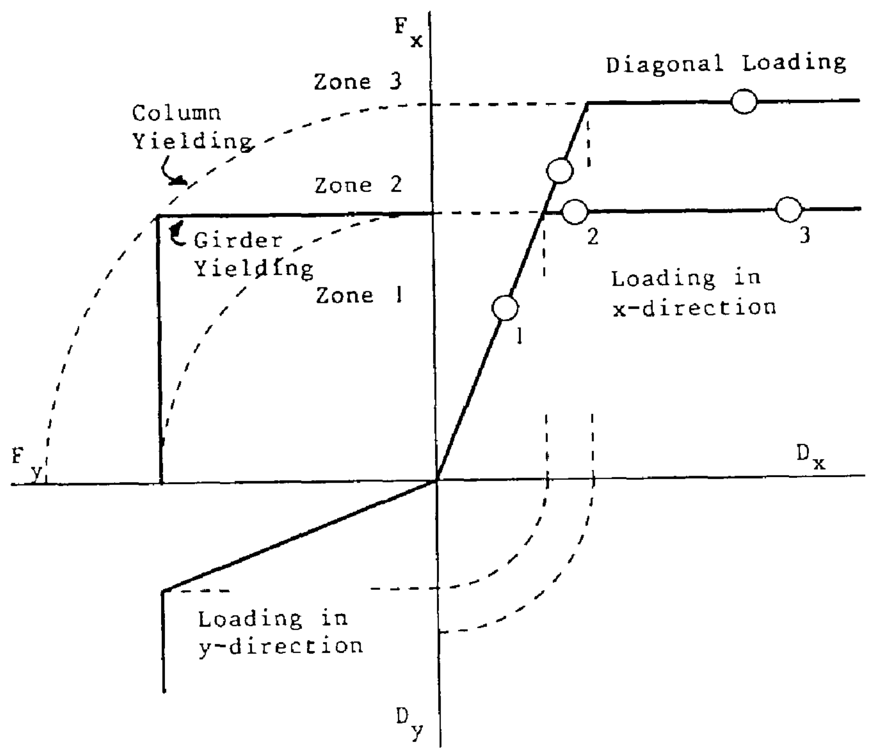


Fig. 16: Effect of Bi-directional Loading

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 super-structure 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

The guidelines is drafted and discussed by a working group with the following members; S. Otani, chairman, M. Teshigawara, Secretary, M. Hayashi, Y. Inoue, T. Kaminosono, I. Kawabata, M. Kimizuka, S. Nakata, K. Yoshioka. The contribution of the working group members is deeply acknowledged. Nonlinear analyses of idealized structures were carried out by Dr. Y. Sakai, Dr. D. Saito, Dr. H. Kinugasa and Mr. K. Fu. Benchmark structures were designed by working group (Mr. N. Kani, chairman) of Japan Structural Consultants Association.

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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