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# Outline of AIJ design guidelines for RC buildings

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ABSTRACT: The Architectural Institute of Japan (AIJ) published "Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings based on Ultimate Strength Concept" in November, 1990, as a first attempt to develop an ultimate strength design procedure for reinforced concrete in Japan. The paper describes the basic concept and introduces the requirements and summary of their commentary.

## 1 INTRODUCTION

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

Building Standard Law was proclaimed in 1950 to regulate all building construction throughout Japan. Structural design method was provided in Building Standard Law Enforcement Order, in which two levels of allowable stresses were introduced for (a) long-term normal loading conditions and (b) short-term unusual (earthquake and wind) loading situations. The allowable stresses for 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.

Building Standard Law Enforcement Order, revised in 1981 (Building Center of Japan 1988), 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. AIJ Guidelines has not been approved for general use by the Ministry of Construction.

Basic philosophy outlined in 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, 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, and 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).

## 2 SCOPE OF GUIDELINES

AIJ Guidelines provides an earthquake resistant design procedure for reinforced concrete moment-resisting frame or (structural) wall-frame structures, regular in shape, but total height not more than 45 m.

The regularity of a building is measured by eccentricity ratio and story stiffness ratio defined in Building Standard Law Enforcing Order (Building Center of Japan 1988). The eccentricity ratio is a ratio of the distance between the centers of rigidity and mass to the elastic radius of gyration (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 story stiffness ratio is the reciprocal $r_{si}$ of an elastic story drift angle at a story i under the design earthquake loads to the algebraic average of $r_{si}$'s. If the eccentricity ratio is less than 0.15, and the story stiffness ratio greater than 0.60, the structure may be judged regular.

Types of concrete by aggregates must be normal weight concrete with design specified strength between 210 and 360 kg/cm$^2$. The use of light weight aggregate concrete is excluded because of the lack of supporting test data. Grades of reinforcing bars must be SD295, SD345 or SD390 (specified yield stresses of 295, 345 or 390 MPa, respectively). Deformed prestressing bars may be used as shear reinforcement.

## 3 DESIGN PRINCIPLES

A structure must be designed to ensure strength and serviceability necessary for gravity loading and for medium intensity earthquake motions, and to maintain the resistance to an expected deformation during a strong earthquake motion. The intensities and acceptable damage levels are:

(a) Moderate earthquake: an earthquake which may occur several times during a building lifetime (max. ground velocity of 150–200 mm/sec). Damage must be restored with minor repair work; i.e., cracking, but no plastic deformation accompanying steel yielding.

(b) Strong earthquake: an earthquake which may possibly occur once in a building lifetime (max. ground velocity of 400–500 mm/sec). Damages must be restored after substantial repair work.

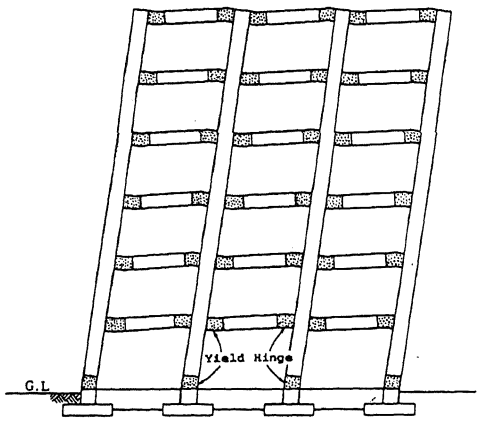
AIJ Guidelines assumes a structure to oscillate dominantly in the fundamental mode during a strong earthquake forming a specified yield mechanism. The yield mechanism must be of total collapse type (Fig.1), which develops plastic hinges throughout a structure and develops a uniform inter–story drift over the entire structural height. A partial yield mechanism (Fig.2), 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, each yield hinge develops much less plastic deformation in a total yield mechanism than in a partial yield mechanism.

A yield mechanism specified for a moment–resisting frames is of beam yield type (Fig.1.a), in which yield hinges develop at the ends of all floor beams and at the base of the first story columns. Yield hinges may form at the following locations; (a) at the top of the top story columns, (b) in external columns when axial loads are reduced to tension by earthquake forces, and (c) in interior columns where gravity load actions govern the design.

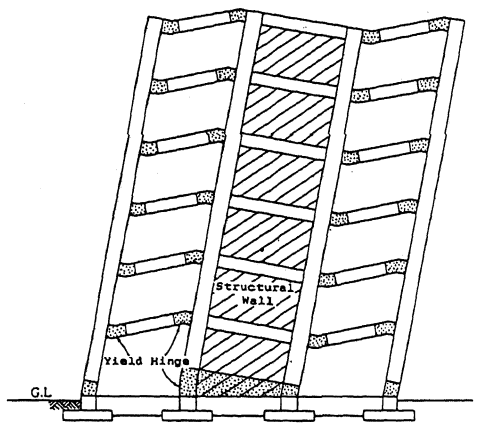
Yielding hinges are desired at beam ends because of (a) ductility and (b) stable energy dissipation. Furthermore, simultaneous yielding at all beam ends can dissipate substantial hysteretic energy as a structure. On the other hands, yielding at column ends may lead to a partial yield mechanism, and a significant damage in a column may lead to the collapse; columns are difficult to develop 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 (Fig.1.b) or by uplifting (Fig.1.c) at the base. Both yield mechanisms are believed to be ductile. With structural walls, the deformation of a structure distributes almost uniformly along the structural height. Therefore, a frame part does not necessarily develop beam yielding, and yield hinges may be planned at column ends. However, the columns are not encouraged to yield because of their limited ductility.

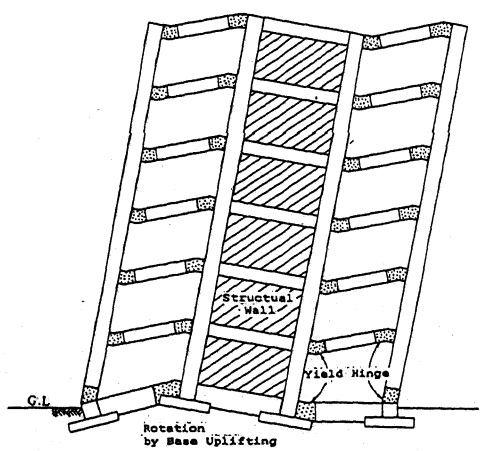
Yield hinges must not be planned in the foundation and piles. However, if a structural wall forms a yield mechanism by uplifting at the base, yield hinges may be planned at foundation girder ends. Damage in the foundation is difficult to inspect and repair after an earthquake. Sound foundation is important to provide a stable support to a super–structure.




(a) Beam yield mechanism




(b) Wall yield mechanism




(c) Wall up–lifting mechanism

Figure 1. Total yield mechanisms

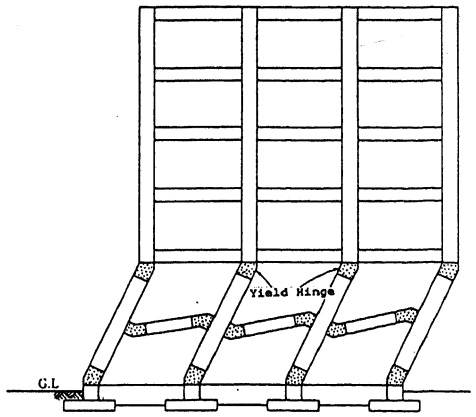



Figure 2. Partial Yield Mechanism

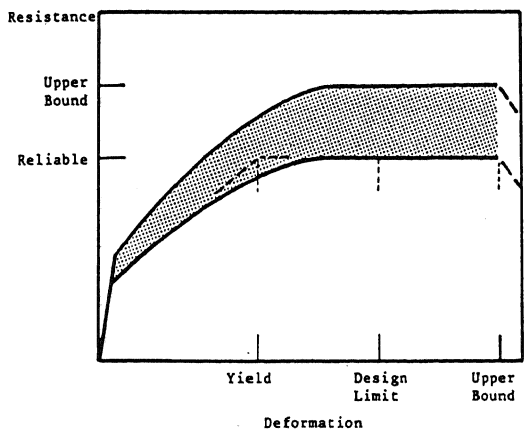



Figure 3. Design load and deformation

The design is carried out in two stages; (a) yield mechanism design and (b) yield mechanism assurance design, both are described in details later.

Three limiting deformations are considered (Fig.3): (a) yield deformation, (b) design limit deformation, and (c) upper bound deformation.

Yield deformation is a deformation of an imaginary yield point on the load–displacement relation as a structure, at which all planned yield hinges are assumed to form simultaneously; in general, yield hinges yield at different deformation levels.

Design limit deformation is a maximum deformation expected during a strong earthquake. AIJ Guidelines assumes a story drift angle of 0.01 rad for the deformation, and requires full specified resistance of a structure to be developed prior to this deformation.

Upper bound deformation is the upper bound earthquake response deformation considering all probable uncertainties such as intensity level and the characteristics of strong earthquakes, including soil effect, the performance of a designed structure, and the method of inelastic dynamic analysis. AIJ Guidelines assumes a story drift angle of 1/66 rad for a frame structure and 1/75 rad for a wall–frame structure. The resistance at the yield hinges must be sustained to this deformation.

## 4. YIELD MECHANISM DESIGN

In yield mechanism design, proper resistance and ductility is provided at planned yield hinges to satisfy the horizontal load carrying capacity and deformation of a structure.

### 4.1 *Design earthquake load*

Design earthquake load may be assumed to act separately in the two principal directions. Minimum design base shear coefficient $C_1$ is given

$$C_1 = Z R_t C_B \quad (1)$$

where, Z: earthquake zone factor, which varies from 0.7 to 1.0; $R_t$: vibration characteristic factor given in Eqs.(2)–(4); and $C_B$: standard base shear coefficient; not less than 0.25 for a frame structure, and 0.30 for a wall–frame structure.

$$R_t = 1 \quad \text{for } T<Tc \quad (2)$$

$$R_t = 1 - 0.2(T/Tc - 1)^2 \quad \text{for } Tc<T<2\,Tc \quad (3)$$

$$R_t = 1.6\,Tc / T \quad \text{for } 2\,Tc<T \quad (4)$$

where, T: fundamental period of the structure; Tc: periods defined for soil conditions, 0.4 sec for stiff soil, 0.6 sec for intermediate soilm and 0.8 sec for soft soil.

A dynamic response amplitude is closely related to the lateral strength. Hence, the design earthquake load should ideally be determined so that the maximum response of a structure during design earthquake motions should be less than the design limit deformation. AIJ Guidelines, however, adapted the zone factor and vibration characteristic factor from the Building Standard Law Enforcement Order.

A concentrated load $P_t$, given in Eq.(5), must be applied at the roof level, and the remaining design earthquake forces must be determined in accordance with seismic coefficients of a triangular distribution;

$$P_t = 0.10\,T\,Q_1 \quad (5)$$

where, T: fundamental period of the structure; and $Q_1$: base shear. $P_t$ may be taken as zero for a wall–frame structure and for a frame structure of not more than 6 stories. The triangular distribution of earthquake loads was selected to simulate the fundamental mode shape. In a tall frame building, however, the triangular design load distribution tends to develop large beam deformation in the upper stories attributable to higher mode oscillation. Therefore, an additional load at the roof is specified for a relatively high–rise building.

### 4.2 *Linear analysis*

A linear structural analysis using realistic stiffness and design earthquake loads could closely approximate the

Table 1. Stiffness reduction factor

| | Flexural Stiffness | Axial Stiffness | Shear Stiffness |
|---|---|---|---|
| (a) Beam | | | |
| (Yield Hinges) | 1.0 | (1.0) | (1.0) |
| (No Hinge) | 0.3–0.5 | (1.0) | 0.3–(1.0) |
| (b) Column | | | |
| (No Hinge) | 1.0 | 1.0 | (1.0) |
| (One Yield Hinge) | 0.7 | 1.0 | (1.0) |
| (Two Yield Hinge) | 0.3–0.5 | 1.0 | (1.0) |
| (c) Structural Wall | | | |
| (No Hinge) | 1.0 | 1.0 | 0.5–1.0 |
| (Yield Hinge) | 0.3–0.5 | 1.0 | 0.3–0.5 |
| (d) Beam–column Connection | | | |
| | --- | --- | 1.0 |
| (e) Slab | | | |
| | 0.0 | (1.0) | (1.0) |

[Note] The stiffness may be taken infinity for 1.0.

stress distribution of a specified yield mechanism. Realistic secant stiffness at the stress level of each member should be used in the analysis (a) to avoid yield hinges forming in a moderate earthquake, (b) to develop simultaneous yielding at the planned yield hinges to prevent a concentration of deformation in a strong earthquake, and (c) to ensure the formation of the total yield mechanism prior to the design limit deformation. For simplicity, the stiffness reduction factor listed in Table 1 may be used.

### 4.3 *Moment redistribution*

Actual flexural strength at a critical section tends to significantly exceed the elastic moment attributable to (a) the rounding of the number of reinforcing bars, (b) the limitation in available bar sizes, and (c) the use of common bar sizes in a story, and (d) the use of reinforcing bars continuous through a beam–column joint. The required amount of top and bottom steel is significantly different at a beam end, the amount of bottom reinforcement being governed by the minimum requirements of a tensile reinforcement ratio or of a compressive to tensile reinforcement ratio. Furthermore, the ultimate flexural strength of a wall base often exceeds the required design moment. If the strength at the yield hinges becomes excessively large, the regions other than at the planned yield hinges must be designed for proportionally higher design stresses. Such design may be uneconomical and impractical.

In order to minimize the overstrength at the planned yield hinges, the redistribution of elastic moments is allowed to optimize the strength distribution and reinforcement arrangement within a structure; i.e., the large elastic moments at some planned yield hinges are redistributed to other planned yield hinges with smaller elastic moments, resulting in a more uniform strength distribution.

If the redistribution is allowed without limitation, the location of reduced elastic moment may develop yielding even from a moderate earthquake and suffer a significant damage from a strong earthquake. Consequently, the maximum amount of redistributed moments is limited to 20 percent in a frame structure and 25 percent in a wall–frame structure. The moments after redistribution should be examined to be in equilibrium with the design earthquake loads. If elastic moments are redistributed between different floors, the sum of the total joint moments at a floor must not deviate from that of the elastic moments by more than 5 percent in a frame structure and 15 percent in a wall–frame structure.

### 4.4 *Deformation limit*

AIJ Guidelines assumes the design limit deformation to be a story drift angle of 1/100 rad. The drift is defined as the deformation at the height of the centroid of the external forces relative to the foundation; the foundation deformation may be ignored because the stiffness is difficult to evaluate. The value of design limit deformation was determined (a) to avoid damage to nonstructural elements, and (b) to ensure human safety and evacuation.

The elastic story drift calculated under design earthquake loads is limited to 1/200. The limitation is intended to ensure a structure to possess adequate stiffness and also to form a specified yielding mechanism prior to the design limit deformation.

The building must maintain the required lateral load resistance to the upper bound deformation; i.e., a story drift angle of 1/66 rad for a frame structure and 1/75 rad for a wall–frame structure. The upper bound deformation of a member should be evaluated by a static nonlinear analysis. For simplicity, beams, columns, structural walls, and beams connected to a wall should be detailed to deform to a member rotational angle of 1/50, 1/67, 1/75, and 1/40 rad, respectively.

## 5 YIELD MECHANISM ASSURANCE DESIGN

Member actions at the formation of a specified yield mechanism may be calculated by a static nonlinear analysis under a design lateral load distribution, but the calculated member forces may be exceeded during a strong earthquake by the following reasons:

(a) Actual strength at yield hinges may be larger than the resistance assumed in the yield mechanism design (upper bound strength),

(b) Lateral load distribution during an earthquake is different from that assumed in the static nonlinear analysis (dynamic effect), and

(c) Columns and structural walls are subjected to earthquake motions in the two orthogonal directions simultaneously (concurrency effect of bi–directional response).

In order to assure the formation of a specified yield mechanism of a structure during a strong earthquake, regions other than the planned yield hinges must be provided with resistances against all modes of failure even under the upper bound actions during the strong earthquake. Therefore, basic design actions in the yield mechanism assurance design should be first evaluated by a static nonlinear analysis under a design earthquake load distribution assuming the upper bound strength at

the planned yield hinges, and then magnified by the dynamic effect and the concurrency effect of bi-directional response.

### 5.1 *Upper bound strength*

Reliable strength and upper bound strength are used in AIJ Guidelines. The reliable strength is the lower bound strength of a section or member calculated using member dimensions and specified material strengths. The upper bound strength is flexural strength at a hinge section evaluated by taking into consideration all possible factors which contribute to the strength enhancement, such as, (a) statistical upper bound material strengths, (b) additional construction steel, (c) reliability of strength evaluation methods, and (d) spread of effective slab width for beams and effective wall width for columns. The effect of strain hardening is not considered in AIJ Guidelines because the member deformation is limited. Note that the upper bound material strength may not be necessary if the same reinforcement is used throughout the structure.

Accoding to a report (Takahashi 1985), the compressive strength fell below the design nominal strength in less than 0.5 % of standard water-cured coupons obtained from construction sites. Therefore, the design specified strength may be used for concrete strength in calculating the reliable strength.

According to a report (Ikeda 1986) on tension tests of reinforcing bars, 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. Therefore, the specified yield strength may be used in calculating the reliable strength. The yield stress in the upper bound strength evaluation may be taken to be 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.

### 5.2 *Dynamic effects*

The nonlinear vibration of a structure, forming a total yield mechanism, is normally dominated by the fundamental mode. Thus, basic design forces are calculated by a static nonlinear analysis using lateral loads of a specified fundamental mode distribution. The upper bound strength must be used at planned yield hinges. A static nonlinear analysis under a monotonically increasing load is desirable. If a specified total yield mechanism is not formed before an upper bound deformation, the horizontal load carrying capacity and reference design forces must be defined at the upper bound deformation.

Because lateral loads during an earthquake distribute in a manner different from the static earthquake loads due to the contribution of higher mode oscillation, 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), maximum higher mode forces during a nonlinear earthquake response were found 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.

The dynamic amplification factor $\omega$ (= $Q_{dmax} / Q_{smax}$) is defined as a ratio of the maximum dynamic story shear $Q_{dmax}$ during an earthquake to the static story shear $Q_{smax}$ at the formation of a specified total yield mechanism. The maximum higher mode story shear $Q_{fmax}$ can be estimated from the intensity of a ground motion. Note that the maximum story shears of the fundamental mode component (= $Q_{smax}$) and the higher mode components (= $Q_{fmax}$) do not occur at the same time, especially (a) in a long-period building, (b) in a frame structure, and (c) in the lower part of a building. However, AIJ Guidelines proposes a dynamic amplification factor by estimating the upper bound of the maximum dynamic story shear $Q_{dmax}$ as a simple sum of the two maxima; hence,

$$\omega = 1.0 + Q_{fmax} / Q_{smax} \tag{6}$$

It should be noted that a smaller amplification factor is required for a stronger structure (large $Q_{smax}$) because $Q_{fmax}$ is constant for a design earthquake motion and a structure.

Although a shear distribution among columns of a story may vary with time during an earthquake, the same 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, defined as a ratio of the maximum response column moment to the reference design moment, 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. If the stiffnesses of walls and columns are comparable, a higher mode story shears should be distributed to columns and walls by ratios $\beta_{chi}$ and $\beta_{whi}$, dependent on the elastic stiffness properties of the members;

$$\beta_{chi} = I_c / (I_c + I_w) \tag{7}$$

$$\beta_{whi} = I_w / (I_c + I_w) \tag{8}$$

where, $I_c$: the total stiffness of the columns under antisymmetric bending in a story, and $I_w$: the total stiffness of the walls under antisymmetric bending in a story (considering shear stiffness).

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 may be used in a wall although design moments may be overestimated in the middle stories.

After simplifying the relation, AIJ Guidelines proposes the dynamic amplification factors for design moment and shear in columns and structural walls as

follows:

$$\omega_{ci} = 1.0 + (\Delta\omega_i / \phi_o)(\beta_{chi} / \beta_{ci}) \quad (9)$$

$$\omega_{wi} = 1.0 + (\Delta\omega_i / \phi_o)(\beta_{whi} / \beta_{wi}) \quad (10)$$

The higher mode coefficient $\Delta\omega_i$ is given by

$$\Delta\omega_1 = 0.25 \quad (11)$$

$$\Delta\omega_i = 0.20 \qquad \text{for } 1 < i < n/2 \quad (12)$$

$$\Delta\omega_i = 0.20 + 0.10\,(i - n/2) \qquad \text{for } i > n/2 \quad (13)$$

where, $\omega_{ci}$, $\omega_{wi}$: dynamic magnification factors of columns and structural walls at i-th story; $\phi_o$: structural strength magnification factor in the yield mechanism assurance design ($=C_{1o} / 0.25$); $C_{1o}$: base shear coefficient in the yield mechanism assurance design; $\beta_{ci}$, $\beta_{wi}$: ratio of shears carried by columns and structural walls in i-th story under fundamental mode distribution of earthquake forces; $\beta_{chi}$, $\beta_{whi}$: ratio of shears carried by columns and structural walls in i-th story under higher mode distribution of earthquake forces; and n: number of stories.

### 5.3 *Concurrency of bi-directional response*

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, assuming the upper bound strength at each yield hinge. However, the probability of concurrent mechanism formation in the two directions is small and the design may be over conservative keeping in mind the use of the upper bound strength at the yield hinges.

AIJ Guidelines proposes to add, to the design member forces (including upper bound strength and dynamic effect) in the direction of concern, 50 % of member actions calculated by a static nonlinear analysis at the formation of a collapse mechanism in the orthogonal direction. In other words, reference design shears and moments of a column must be magnified by a factor equal to the sum of a dynamic magnification factor and a concurrency safety factor (= 0.10). However, design axial load of columns and structural walls must be calculated by adding 50 percent of the axial load caused by orthogonal earthquake forces.

## 6 CONCLUSIONS

The design concept used in AIJ Guidelines is briefly outlined. A building should develop a specified total yield mechanism under a design earthquake, forming yield hinges at locations easy to ensure deformation capacity. The locations, where yield hinges are not intended, are to be provided with sufficient resistance. The application is limited to a regular frame and wall-frame building of not more than 45 m in height.

## ACKNOWLEDGMENT

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