ULTIMATE STRENGTH DESIGN GUIDELINES

FOR

REINFORCED CONCRETE BUILDINGS

by

PRESSS GUIDELINES WORKING GROUP

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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.

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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		Short-term Loading
	Top Reinforcement	Bottom Reinforcement	
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)

Steel Grade	Long-term Loading Tension Compres.	Long-term Loading Shear Reinf.	Short-term Loading Tension Compres.	Short-term Loading Shear Reinf.	Material Strength Tension Compres.	Material Strength 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

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

$$C_i = Z R_i A_i C_B \tag{4.2}$$

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,

$$Nu = Ac\ Fc \tag{4.6}$$

$$Nt = Ag\ fy \tag{4.7}$$

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

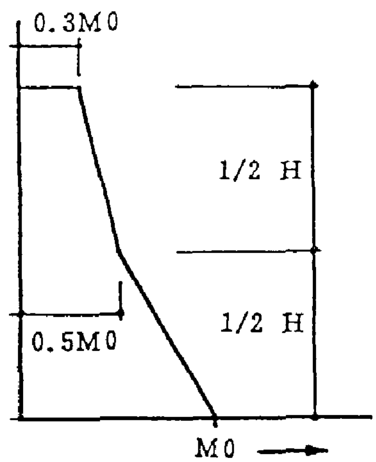
Design actions at a PCa connection shall be evaluated by the existing forces calculated at design proof deformation magnified by the amplification factors specified in Section 4.3.2. Design of PCa connections shall conform to the provisions of "Design Manual for PCa Connections."

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

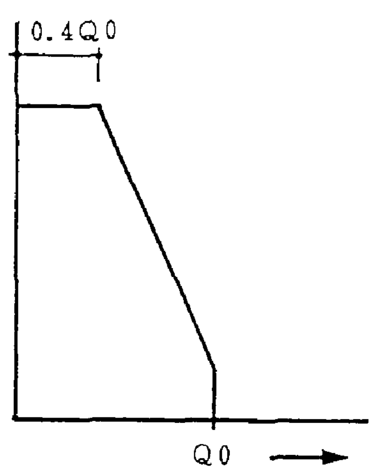
Member Action	Amplification Factor
(a) Girders	
Shear	a_1
(b) Columns	
Bending moment	a_2
Shear	a_3
(c) Structural Walls	
Bending moment	a_4
Shear	a_5

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):

$$Q_{uni} = C_{uni} W_i \qquad (5.1)$$

$$C_{uni} = Z R_t A_i C_{uns} \qquad (5.2)$$

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.