AIJ Standards for Structural Calculation of Steel Reinforced Concrete Structures
(1987)

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Preface

Steel reinforced concrete (SRC) construction has been used widely for building structures in Japan, and the Standards for Structural Calculation of Steel Reinforced Concrete Structures were published first in 1958 by Architectural Institute of Japan (AIJ). The Standards have been revised several times since then, and the English version presented here is based on the latest edition revised in 1987.

The structural calculation of SRC structures is mainly based on the allowable stress design method, but design formulas to estimate the ultimate strength of SRC members and connections are also provided since some structures are required to be examined for their lateral load resisting capacity against earthquakes. The superposed strength method has been used for calculating the strength of SRC members since the outset of the Standards. The range of application of this method has been expanded after each revision, and in the 1975 edition, this method was employed for designing connections, joints, column-bases and structural walls in addition to primary members.

In the latest revision (1987), the design formulas that were shown in the commentary of the previous edition were incorporated into the Standards, thus the volume increased significantly. The English version includes only a brief commentary, which puts emphasis on parts deemed important for those who are not familiar to the Japanese SRC Standards. A detailed commentary is attached in the Japanese version.

Architectural Institute of Japan

September, 1991

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The manuscript of the Brief Commentary was prepared in Japanese by Minoru Wakabayashi. The Standards and the Brief Commentary were translated into English by Shosuke Morino, which were corrected by Shunsuke Otani and Masayoshi Nakashima. The work for the English translation was supported by Kozai Club.

AIJ STANDARDS FOR STRUCTURAL CALCULATION OF STEEL REINFORCED CONCRETE STRUCTURES AND BRIEF COMMENTARY

(1987)

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AIJ Standards for Structural Calculation of Steel Reinforced Concrete Structures

(1987)

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Chapter 1 General

Article 1 Scope

The standards shall be applied to the structural calculation of steel reinforced concrete structures and composite steel tube-concrete structures, except for those cases where the structural design is based on special investigations.

Article 2 Procedure of Structural Calculation

1. The structural calculation shall be based on the method of allowable stress design. However, story drift angle, stiffness, eccentricity and horizontal shear strength shall be properly considered in order to ensure an adequate level of safety under earthquake conditions, depending upon the scale and the shape of the structure. In addition, the serviceability must be assured against deformation of members or vibration.

2. In the allowable stress design, the long-term and the short-term stress resultants in a member are determined from the combination of the stresses caused by the loads and external forces, and shall not exceed the member resistance at the long-term and the short-term allowable stresses.

3. The horizontal load resistance of each story shall be greater than the required strength.

Article 3 Notation

Notations for the main symbols used in the standards are as follows:

A : gross cross-sectional area of section (cm^2)
a : partial area of section (cm^2)
B : width of T-section (cm)
b : width of member (cm)
D : depth or diameter of member (cm)
d : partial depth of member (cm)
E : modulus of elasticity (kg/cm^2)
F : strength (kg/cm^2)

h : height (cm)
I : moment of inertia (cm^4)
j : distance between compressive and tensile resultants (cm)
l : length or span (cm)
M : bending moment (kg·cm)
N : axial force (positive in compression) (kg)
n : ratio of moduli of elasticity of steel to concrete
P : concentrated force (kg)
p : reinforcement ratio or steel ratio
Q : shear force (kg)
R : horizontal shear strength of frame (kg)
r : reduction factor
S : moment of area (cm^3)
T : tension force (kg)
t : thickness (cm)
V : volume (cm^3)
x : distance or interval (cm)
Y : yield ratio
Z : section modulus (cm^3)
α : coefficient as a function of shear-span ratio ($M/(Q\cdot d)$)
β : modification factor
γ : weight for unit volume (t/m^3) or ratio of compression reinforcement
δ : amplification factor
η : depth-width ratio
θ : angle
μ : limitation factor
ν : factor of safety
σ : axial stress (kg/cm^2)
τ : shear stress (kg/cm^2)
ϕ : perimeter of reinforcing bar

Notations for main subscripts indicating materials and parts are as follows:
c : concrete
m : main reinforcement
r : reinforced concrete
s : steel
w : web reinforcement

Chapter 2 Materials

Article 4 Quality, Shapes and Dimensions of Steel

1. Unless otherwise provided, structural steel materials shall be of the quality specified in the applicable standards listed in Table 1.

Table 1 Standards for Quality of Structural Steel Materials

Numbers	Titles
JIS G 3101	Rolled Steel for General Structure : SS400, SS490 and SS540
JIS G 3106	Rolled Steel for Welded Structure : SM400 A, B and C ; SM490 A, B and C ; SM490Y A, B and C ; SM520 B and C ; and SM570
JIS G 3114	Hot-Rolled Weather Steel for Welded Structure : SMA400 and SMA490
JIS G 3444	Carbon Steel Tubes for General Structural Purposes : STK400 and STK490
JIS G 3466	Carbon Steel Square Pipes for General Structural Purposes : STKR400 and STKR490
JIS G 5201	Centrifugal Casting Steel Tube for Welded Structure : SCW490-CF and SCW520-CF
JIS B 1186	Sets of High-Strength Hexagon Bolt, Hexagon Nut and Plain Washers for Friction Grip Joints
JIS Z 3211	Covered Electrodes for Mild Steel
JIS Z 3212	Covered Electrodes for High-Tensile Strength Steel
JIS Z 3311	Steel Wire for Submerged Arc Welding
JIS G 5101	Carbon Steel Castings
JIS G 3201	Carbon Steel Forgings

2. Unless otherwise provided, structural steel materials shall be of the shapes and dimensions prescribed in Table 2.

Table 2 Standards for Shapes and Sizes of Structural Steel Materials

Numbers	Titles
JIS G 3192	Dimensions, Weight and Permissible Variations of Hot-Rolled Steel Sections
JIS G 3193	Dimensions, Weight and Permissible Variations of Hot-Rolled Steel Plates, Sheets and Strip
JIS G 3194	Shape, Dimensions, Weight and Tolerance for Hot-Rolled Flat Steel
JIS G 3191	Shape, Dimensions, Weight and Tolerance for Hot-Rolled Steel Bar and Bar-in-Coil
JIS G 3444	Carbon Steel Tubes for General Structural Purposes
JIS G 3466	Carbon Steel Square Pipes for General Structural Purposes
JIS B 1186	Sets of High-Strength Hexagon Bolts, Hexagon Nut and Plain Washers for Friction Grip Joints
JIS B 1180	Hexagon Head Bolts
JIS B 1181	Hexagon Nuts

Article 5 Quality, Shapes and Sizes of Reinforcing Bars

Unless otherwise provided, reinforcing bars shall be of quality specified in JIS G3112 (steel bars for reinforced concrete structure) and JIS G 3117 (rerolled steel bars for reinforced concrete structure). When the specified minimum yield stress of reinforcing bars greater than 30 kg/mm² is used, the design standard strength of concrete shall conform to Table 3.

Table 3 Combination of Reinforcing Bars and Concrete

Grade of reinforcing bars	Design standard strength F_c (kg/cm²)
SR295, SD295 A and B SRR390, welded wire mesh	not less than 150
SD345, SD390	not less than 180

Article 6 Class and Quality of Concrete

1. Class of concrete according to aggregate used shall conform to Table 4.

2. Quality of concrete shall conform to the requirements of Section 2 in JASS 5 (reinforced concrete work).

Table 4 Class, Design Standard Strength and Range of Dry Weight for Unit Volume of Concrete

Class of concrete		Aggregate: Coarse(1)	Aggregate: Fine(2)	Design standard strength (kg/cm²)	Dry unit weight (t/m³)
Ordinary concrete		Gravel Crushed stone Blast-furnace slag	Sand Crushed sand Slag sand	150 180 210 225 240	2.2-2.4 (as a standard)
Light-weight concrete	1st	Artificial light-weight aggregate	(ditto)	150 180 210 225	1.7-2.0
	2nd	(ditto)	Artificial light-weight fine aggregate or those partly replaced by sand, crushed sand or slag sand	150 180 210 225	1.4-1.7

(1) Including mixed use of gravel, crushed stones and blast-furnace slag.
(2) Including mixed use of sand, crushed sand and blast-furnace slag sand.

Chapter 3 Detailed Structural Requirements

Article 7 Detailed Structural Requirements

1. Main Reinforcing Bars

(1) Main reinforcing bars shall be not less than of 13mm in diameter for plain bars, nor of size D13 for deformed bars.

(2) Clear distance between parallel bars shall be not less than 1.25 times the nominal maximum size of coarse aggregate, nor 25mm, nor 1.5 times the diameter of plain bars or the nominal diameter of deformed bars. Clear distance between longitudinal bars and steel section shall be not less than 25mm nor 1.25 times the nominal maximum size of coarse aggregate.

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2. Stirrups and Hoops

(1) Stirrups and hoops shall be not less than of 9mm in diameter for plain bars, nor of size D10 for deformed bars.

(2) Spacing of stirrups and hoops shall conform to the requirements below:

(i) Spacing of stirrups shall not exceed half the beam depth and 25cm, when plain bars of 9mm diameter or deformed bars of size D10 are used. In the case where stirrups of the larger diameter are used, the spacing may be increased up to half the depth and 45cm.

(ii) Spacing of hoops shall not exceed 10cm, when plain bars of 9mm diameter or deformed bars of size D10 are used. Outside the length equal to 1.5 times the maximum dimension of a column, the spacing may be increased to 1.5 times the value specified above. In the case where hoops of the larger diameter are used, the spacing may be increased up to 20cm.

(3) Reinforcement ratio of stirrups or hoops, defined by Eq. (1), shall conform to the requirements below:

$$\left.\begin{array}{ll} {}_{w}p=\dfrac{{}_{w}a}{b\cdot x} & \text{(general steel reinforced concrete structure)} \\ {}_{w}p=\dfrac{{}_{w}a}{b'\cdot x} & \text{(concrete encased, and concrete encased-filled steel tubular structures)} \end{array}\right\} \quad (1)$$

(i) columns encased by open-web type steel : not less than 0.2%

(ii) columns encased by full-web type steel of open section : not less than 0.1%.

(iii) concrete encased, and concrete encased-filled steel tubular columns: not less than 0.2%.

(4) Stirrups and hoops shall be tightly connected to longitudinal reinforcing bars, and shall enclose all longitudinal steel elements to confine the concrete inside the longitudinal steel elements.

(5) Clear distance between stirrups or hoops and a steel section shall be not less than 2.5cm.

3. Steel

(1) Width-thickness ratio of a plate element of a steel section and diameter-thickness ratio of a steel tubular section shall be limited in order to assure a sufficient deformation capacity.

(2) When a built-up steel is used, longitudinal members shall be connected each other by batten plates, lacing members or other connecting members. Distance between connecting members shall be limited in order to assure a sufficient

deformation capacity.

4. Beams

(1) Longitudinal reinforcing bars shall not be placed in more than two layers in a steel reinforced concrete or a reinforced concrete beam except in special cases.

(2) Diameter of a hole in a steel reinforced concrete beam shall be not greater than 0.4 times the depth of the total section, and not greater than 0.7 times the depth of an encased steel section.

5. Columns and Compression Members

(1) Cross-sectional area of longitudinal steel members in a column or a compression member shall be not less than 0.8% of the gross area of concrete.

(2) Effective buckling length of a column shall not be more than 30 times the smallest dimension of the cross section.

(3) Effective buckling length of a compression member shall be not more than 50 times the smallest dimension of the cross section.

6. Beam-to-Column Connections

(1) Steel connection shall be detailed in order to reduce the local stresses, and to make the work of reinforcing bar arrangement and concrete casting efficient.

(2) Longitudinal reinforcing bars shall be anchored to develop full strength.

(3) Reinforcing bars in a beam-to-column connection shall not pass through steel plates, if possible. Where the forming of holes in the steel plates is inevitable for the bar arrangement, the amount of holes shall be kept to a minimum.

7. Joints

Joints of steel members and splices of reinforcing bars shall, in general, not be located either at the points of large stresses or at a concentrated point.

8. Column Bases

Anchor bolts and a base plate necessary for the construction shall be placed at the base of a steel column.

9. Slabs

(1) Structural details of a reinforced concrete slab cast in situ using ordinary forms shall conform to Standards for Structural Calculation of Reinforced Concrete Structures.

(2) Concrete slab cast in situ using deck plates shall conform to the requirements below:

(i) A slab without shear connectors between concrete and a deck plate is treated as a reinforced concrete slab neglecting the deck plate, and the structural details for such a slab shall conform to Standards for Structural Calculation of Reinforced Concrete Structures.

(ii) A slab with shear connectors between concrete and a deck plate can be treated as a composite slab, and the structural details for such a slab shall conform to Design Recommendations for Composite Constructions, Part 2 : Design Recommendations for Composite Slabs Made of Deck Plate and Concrete.

(iii) Deck plate shall be connected to surrounding beams by welding or high-strength bolts.

(3) Reinforcing bars in a reinforced concrete slab shall be anchored in the surrounding members or adjacent slabs without being obstructed by steel members.

10. Structural Walls and Braces

(1) Structural details of reinforced concrete structural walls shall conform to Standards for Structural Calculation of Reinforced Concrete Structures.

(2) In the case of a structural wall containing a steel brace, such as a steel flat bar, reinforcement shall be placed in order to restrain the local buckling of the brace.

(3) In the case of a structural wall containing steel plate, reinforcement shall be placed on both sides of the plate.

(4) Steel reinforced concrete braces and composite steel tube-concrete braces shall conform to Items 1 through 3 and 5 specified above.

11. Footings

Structural details of a footing shall conform to Standards for Structural Calculation of Reinforced Concrete Structures.

12. Thickness of Covering Concrete

(1) Thickness of concrete covering reinforcement shall conform to JASS 5.

(2) Thickness of concrete covering steel members shall be not less than 5cm, and normally shall be not less than 10cm.

Chapter 4 Allowable Stress Design

Section 1 General

Article 8 Scope

Requirements in this chapter apply to the allowable stress design. The requirements for the structural calculation of composite steel tube and concrete structures are specified in Chapter 6.

Article 9 Loads and Their Combination

1. Loads, external forces and their combination for the structural calculation shall conform to Building Standard Law Enforced Order and Notation of Ministry of Construction, in general.

2. Weight of steel reinforced concrete shall be computed for the actual conditions. However, the values in Table 5 may be used when special investigation is not made.

Table 5 Weight of Steel Reinforced Concrete (t/m^3)

Class of concrete		Weight of steel reinforced concrete
Ordinary concrete		2.5
1st class light weight concrete	$F_c \geq 200$	2.1
	$F_c < 200$	2.05
2nd class light weight concrete	$F_c \geq 200$	1.8
	$F_c < 200$	1.75

Note : The value for 2nd class light weight concrete shall be properly increased with the use of sand, crushed sand or slag sand added to fine light weight aggregate.

Article 10 Material Coefficients

Coefficients of steel, reinforcing bars and concrete shall conform to Table 6. Young's modulus of concrete shall be properly reduced, when the effect of creep under the long-term loading condition is considered.

Table 6 Coefficients of Materials

Material	Young's modulus (kg/cm²)	Shear modulus of elasticity (kg/cm²)	Poisson's ratio	Coefficient of thermal expansion (1/°C)
Steel and reinforcing bar	2.1×10^6	0.81×10^6	0.3	1×10^{-5}
Concrete	$2.1\times10^5 \times ({}_c\gamma/2.3)^{1.5} \times\sqrt{F_c/200}$	$0.9\times10^5 \times ({}_c\gamma/2.3)^{1.5} \times\sqrt{F_c/200}$	1/6	1×10^{-5}

Article 11 Evaluation of Member Stiffness

1. Member stiffness for the calculation of stress and deformation shall be evaluated on the basis of elastic theory. However, the stiffness shall be properly reduced where the stresses are large.

2. Member stiffness for the calculation of stress and deformation may be evaluated on the basis of the gross section of concrete, if the effect of steel materials is small.

3. Effective widths of a T-beam and a member with T-section such as a column with walls shall conform to Standards of Structural Calculation of Reinforced Concrete Structures.

4. Flexural stiffness of a composite beam shall conform to Design Recommendations for Composite Constructions.

5. Rigid portion with a properly evaluated length shall be considered for a beam-to-column connection, a haunched member, columns adjacent to spandrel wall and sagging wall, and a beam adjacent to wing wall. However, where the effect of rigid portion is small, the stresses calculated by neglecting this effect shall be properly increased.

Article 12 Calculation of Stress and Deformation

1. Stress and deformation of a frame may be calculated by considering only the flexural deformation of each member.

2. Stress and deformation of a frame subjected to gravity loads may be calculated on the following assumptions:

(1) Each frame which is an element of a building structure may be analyzed as an independent planner frame.

(2) Horizontal joint displacements may be neglected in a symmetrical building, or in a building with properly arranged structural walls.

3. Stress and deformation of a frame subjected to horizontal forces may be calculated on the following assumptions :

(1) Horizontal forces act in the two orthogonal directions independently.

(2) Horizontal forces are concentrated at the floor levels. Where the effect of horizontal forces acting at intermediate points of a story is large, the effect shall be considered.

(3) Where the effect of torsion due to earthquake loading is considered, the point of application of the story shear force may be taken at the center of gravity above the story under consideration.

(4) Floors may be considered, in general, to be perfectly rigid in plane.

Article 13 Stress during Construction

Where the stresses during construction are carried by steel members, the safety shall be examined for loads and external forces under actual conditions during the construction. Moreover, stress and deformation during construction shall not be neglected.

Article 14 Allowable Stresses

1. Allowable stresses for steel shall conform to the requirements below.

(1) Allowable stresses for steel shall conform to Table 7, where the values of F shall conform to Table 8.

(2) Allowable stresses for other loading conditions shall conform to Design Standards for Steel Structures.

(3) Allowable stresses for high-strength bolts and anchor bolts shall conform to Table 9. Allowable stresses shall be computed for the body area. Design bolt tension for the high-strength bolt shall conform to Table 10.

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Table 7 Allowable Stresses for Steel (kg/cm²)

Long-term loading		Short-term loading
Tension, compression and flexure	Shear	
$F/1.5$	$F/(1.5\sqrt{3})$	1.5 times the values for the long-term loading condition

Table 8 Values of F (kg/cm²)

		Steel for general structures		Steel for welded structures		
		SS400 STK400 STKR400	SS490*	SM400 SMA400	SM490, SM490Y SMA490, STK490 STKR490, SCW490-CF	SM520 SCW520-CF
F	For thickness not larger than 40mm	2400	2800	2400	3300	3600
	For thickness larger than 40mm	2200	2600	2200	3000	3400

Note : *not usable for tubes

Table 9 Allowable Stresses for High-Strength Bolts (F 10 T) and Anchor Bolts (kg/cm²)

Material	Long-term loading		Short-term loading
	Tension	Shear	
High-strength bolts	3100	1500	1.5 times the values for
Anchor bolts	$0.5\,F$	$0.38\,F$	long-term loading condition

Table 10 Design Bolt Tension for High-Strength Bolt (F 10 T) (kg)

Nominal size	M 16	M 20	M 22	M 24
Design Bolt Tention	10600	16500	20500	23800

(4) Allowable stresses on the throat of arc weld may be taken as specified below, provided that welding is performed under sufficient quality control with the use of appropriate welding electrodes for the type of steel. However, welds in SS490 steel shall not be allowed to carry any stresses.

(i) Allowable stresses for fillet welds and partial-penetration welds shall be taken as equal to allowable shear stresses for the base metal to be welded.

(ii) Allowable stresses for butt welds shall be taken as equal to allowable stresses for the base metal to be welded.

(iii) Where steels of different qualities are welded, allowable stresses for welds shall be taken as the smaller of allowable stresses for the base metals.

2. Allowable stresses for reinforcing bars shall conform to Table 11.

Table 11 Allowable Stresses for Reinforcing Bars (kg/cm²)

	For long-term loading		For short-term loading	
	Tension and compression	Shear reinforcement	Tension and compression	Shear reinforcement
SR235, SRR235 SR290, SRR390	1600 1600	1600 2000	2400 3000	2400 3000
SD235, SDR235 SD290 SD340 SD390	1600 2000 2200 (2000) 2200 (2000)	1600 2000 2000 2000	2400 3000 3500 4000	2400 3000 3500 4000
Welded wire mesh	2000	2000	—	3000

Note : The allowable unit stress of bars not less than D29 shall be the value enclosed in parentheses.

3. Allowable stresses for concrete shall conform to Table 12.

4. Allowable bond stresses between reinforcing bar or steel and concrete shall conform to Table 13.

(1) Top bars shall be defined as horizontal bars in flexural members placed so that more than 30cm of concrete is cast below.

(2) Allowable bond stresses for deformed bars covered by concrete with thickness less than 1.5 times the bar diameter shall be the values in this table multiplied by

Table 12 Allowable Stresses for Concrete (kg/cm²)

	Long-term loading			Short-term loading		
	Compression	Tension	Shear	Compression	Tension	Shear
Ordinary concrete	$F_c/3$	—	$F_c/30$ but not more than $(5+F_c/100)$	2 times the values for long-term loading condition	—	1.5 times the values for long-term loading condition
1st & 2nd class light weight concrete	$F_c/3$	—	0.9 times the values for ordinary concrete	2 times the values for long-term loading condition	—	1.5 times the values for long-term loading condition

Table 13 Allowable Bond Stresses Between Reinforcing Bar or Steel and Concrete (kg/cm²)

		Long-term loading		Short-term loading
		Top bars	Other bars or steel	
Plain bars		$(4/100)F_c$ but not more than 9	$(6/100)F_c$ but not more than 13.5	1.5 times the values for long-term loading condition
Deformed bars		$F_c/15$ but not more than $(9+(2/75)F_c)$	$F_c/10$ but not more than $(13.5+F_c/25)$	
Steel shapes Steel plates Outside tubes		—	$(2/100)F_c$ bur not more than 4.5	
Inside tubes	Circular	—	1.5	
	Rectangular		1.0	

the ratio of the covering thickness to 1.5 times the bar diameter.

(3) For steel shape and steel plate, portion where placing concrete is rather difficult shall be omitted from the effective bond area.

(4) Allowable bond stresses for embossed steel shape or steel tube embossed inside and cast iron tube shall be properly determined according to the actual conditions.

5. Allowable shear stresses for stud connectors shall be of value for stud connectors in reinforced concrete slabs of uniform thickness, specified in Design Recommendations for Composite Constructions, Part 1: Design Recommendations

for Composite Beams, multiplied by 1/3 for the long-term loading condition, or by 2/3 for the short-term loading condition.

Section 2 Calculations for Structural Members

Article 15 General Assumptions

1. Strengths of steel reinforced concrete members, composite steel tube and concrete members, and their connections shall be taken as the sum of strengths of composing steel, steel tube, reinforced concrete, and filled concrete portions. However, the allowable shear strength of a column or a beam-to-column connection for the long-term loading condition shall be taken as the cracking strength of reinforced concrete portion, except for a concrete filled steel tubular column. The following assumptions shall be used in calculstion:

(1) Local buckling of steel elements and steel tubes do not occur.

(2) The calculation of flexural strength of reinforced concrete portion shall conform to Standards for Structural Calculation of Reinforced Concrete Structures.

2. The strength of a beam or a column with open web of batten plate steel may be calculated by treating the steel member as reinforcing bars in accordance with Standards for Structural Calculation of Reinforced Concrete Structures.

Article 16 Calculation for Flexural Strength of Beams

1. Flexural strength for beams shall be as given by Eq. (2).

(1) $M \leqq {}_sM_0 + {}_rM_0$ (kg·cm) (2)

(2) ${}_sM_0$ shall be as given by Eq. (3).

$${}_sM_0 = {}_sZ \cdot {}_sf_b \quad \text{(kg·cm)} \qquad (3)$$

where ${}_sZ$ shall be taken as the smaller of the section modulus of tension side computed for net section taking into consideration bolt hole reduction, and the section modulus of compression side.

${}_sM_0$ for a lattice beam of symmetrical cross section shall be computed by Eq. (4).

$${}_sM_0 = {}_sa_t \cdot {}_sf_t \cdot {}_sd \quad \text{(kg·cm)} \qquad (4)$$

Flexural resistance of chords of a batten-plate type beam or a truss beam with large eccentricity shall be taken into account in the computation of ${}_sM_0$.

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(3) ${}_{r}M_0$ may be computed by Eq. (5), when the tension reinforcement ratio is not larger than the balanced reinforcement ratio of the beam.

$${}_{r}M_0 = {}_{m}a_t \cdot {}_{m}f_t \cdot {}_{r}j \qquad \text{(kg·cm)} \qquad (5)$$

where ${}_{r}j = (7/8)\,{}_{r}d$.

When the tension reinforcement ratio exceeds the balanced reinforcement ratio, the computation of ${}_{r}M_0$ shall conform to Standards for Structural Calculation of Reinforced Concrete Structures. The tension reinforcement ratio shall be defined by Eqs. (6) and (7).

Rectangular beam : $${}_{m}p_t = \frac{{}_{m}a_t}{b \cdot {}_{r}d} \qquad (6)$$

T-beam : $${}_{m}p_t = \frac{{}_{m}a_t}{B \cdot {}_{r}d} \qquad (7)$$

The balanced reinforcement ratio shall be defined by Eqs. (8) and (9).

Rectangular beam : $${}_{m}p_{tb} = \frac{1}{2\left(1+\frac{{}_{m}f_t}{n \cdot f_c}\right)\left\{\frac{{}_{m}f_t}{f_c}\left(1+\gamma\frac{{}_{r}d_c}{{}_{r}d}\right)-n\cdot\gamma\left(1-\frac{{}_{r}d_c}{{}_{r}d}\right)\right\}} \qquad (8)$$

T-beam : $${}_{m}p_{tb} = \frac{1}{2}\left\{t_1(2-t_1)\frac{f_c}{{}_{m}f_t} - \frac{t_1^2}{n}\right\} \qquad (9)$$

2. In addition to Eq. (2), main girders shall satisfy according to Eq. (30) of Article 17, Item 2.

Article 17 Calculation for Axial and Flexural Strengths of Columns

1. Calculation for columns with symmetrical cross section subjected to combined axial force and bending moment shall be made by Eqs. (10) through (15).

(1) Calculation may be made by equations in either (i) or (ii) below.

(i) (a) When ${}_{r}N_t \leqq N \leqq {}_{r}N_c$ or $M \geqq {}_{s}M_0$

$$\left.\begin{array}{ll} N = {}_{r}N & \\ M \leqq {}_{s}M_0 + {}_{r}M & \text{(kg)} \\ & \text{(kg·cm)} \end{array}\right\} \qquad (10)$$

(b) When $N > {}_{r}N_c$ or $M < {}_{s}M_0$

$$\left.\begin{array}{ll} N \leqq {}_{r}N_c + {}_{s}N & \text{(kg)} \\ M = {}_{s}M & \text{(kg·cm)} \end{array}\right\} \qquad (11)$$

(c) When $N < {}_{r}N_t$ or $M < {}_{s}M_0$ with tensile axial force.

$$\left.\begin{array}{ll} N \geqq {}_{r}N_t + {}_{s}N & \text{(kg)} \\ M = {}_{s}M & \text{(kg·cm)} \end{array}\right\} \qquad (12)$$

(ii) (a) When ${}_{s}N_t \leqq N \leqq {}_{s}N_c$ or $M \geqq {}_{r}M_0$

$$\left.\begin{array}{ll} N = {}_{s}N & \text{(kg)} \\ M \leqq {}_{r}M_0 + {}_{s}M & \text{(kg·cm)} \end{array}\right\} \qquad (13)$$

(b) When $N > {}_{s}N_c$ or $M < {}_{r}M_0$

$$\left.\begin{array}{ll} N \leqq {}_{s}N_c + {}_{r}N & \text{(kg)} \\ M = {}_{r}M & \text{(kg·cm)} \end{array}\right\} \qquad (14)$$

(c) When $N < {}_{s}N_t$ or $M < {}_{r}M_0$ with tensile axial force.

$$\left.\begin{array}{ll} N \geqq {}_{s}N_t + {}_{r}N & \text{(kg)} \\ M = {}_{r}M & \text{(kg·cm)} \end{array}\right\} \qquad (15)$$

(2) ${}_{s}N_c$, ${}_{s}N_t$, ${}_{s}M_0$, ${}_{c}N$ and ${}_{s}M$ shall be computed by Eqs. (16) through (20).

$${}_{s}N_c = {}_{s}A \cdot {}_{s}f_c \qquad \text{(kg)} \qquad (16)$$

$${}_{s}N_t = -{}_{s}A_e \cdot {}_{s}f_t \qquad \text{(kg)} \qquad (17)$$

$${}_{s}M_0 = {}_{s}Z \cdot {}_{s}f_t \qquad \text{(kg·cm)} \qquad (18)$$

For ${}_{s}N$ in compression

$$\frac{{}_{s}N}{{}_{s}A} + \frac{{}_{s}M}{{}_{s}Z} = {}_{s}f_c \qquad \text{(kg·cm)} \qquad (19)$$

For ${}_{s}N$ in tension

$$\frac{{}_{s}N}{{}_{s}A_e} - \frac{{}_{s}M}{{}_{s}Z} = -{}_{s}f_t \qquad \text{(kg·cm)} \qquad (20)$$

where ${}_{s}Z$ and ${}_{s}A_e$ shall be computed for net section considering bolt hole reduction. ${}_{s}Z$ for a lattice column may be as given by ${}_{s}Z = {}_{s}a_t \cdot {}_{s}d$.

Flexural resistance occurring in chords of a batten-plate type column or a truss column with large eccentricity shall be taken into account for the computation of ${}_{s}M_0$, ${}_{s}M$ and ${}_{s}N$.

(3) ${}_{r}N_c$ and ${}_{r}N_t$ shall be as given by Eqs. (21) through (23).

$${}_{r}N_c = \min({}_{r}N_{c1}, {}_{r}N_{c2}) \qquad \text{(kg)} \qquad (21)$$

$$\left.\begin{array}{ll} {}_{r}N_{c1} = A_e \cdot f_c' & \text{(kg)} \\ {}_{r}N_{c2} = \dfrac{A_e \cdot {}_{m}f_c}{n} & \text{(kg)} \end{array}\right\} \qquad (22)$$

$${}_{r}N_t = -{}_{m}A \cdot {}_{m}f_t \qquad \text{(kg)} \qquad (23)$$

(4) ${}_{r}N$ and ${}_{r}M$ shall be as given according to Standards for Structural Calculation of Reinforced Concrete Structures. For columns with rectangular cross section with symmetrically placed bars, ${}_{r}N$ and ${}_{r}M$ shall be as given by Eqs. (24) through (28).

(i) For the case controlled by concrete

(a) When $0 \leqq x_{n1} \leqq 1$

$$\left.\begin{array}{l} {}_{r}n = \dfrac{{}_{r}N}{bD \cdot f_c'} = \dfrac{x_{n1}}{2} + n \cdot {}_{m}p_t\left(2 - \dfrac{1}{x_{n1}}\right) \\ {}_{r}m = \dfrac{{}_{r}M}{bD^2 \cdot f_c'} = \dfrac{x_{n1}}{12}(3 - 2x_{n1}) + \dfrac{n \cdot {}_{m}p_t}{2x_{n1}}(1 - 2\,{}_{r}d_{t1})^2 \end{array}\right\} \qquad (24)$$

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(b) When $x_{n1}>1$

$$\left.\begin{aligned} {}_rn&=\frac{{}_rN}{bD\cdot f_c'}=(1+2n\cdot{}_mp_t)\left(1-\frac{1}{2x_{n1}}\right)\\ {}_rm&=\frac{{}_rM}{bD^2\cdot f_c'}=\frac{1}{2x_{n1}}\left\{\frac{1}{6}+n\cdot{}_mp_t(1-2{}_rd_{t1})^2\right\}\end{aligned}\right\}\quad(25)$$

(ii) For the case controlled by reinforcing bars in compression

$$\left.\begin{aligned} \frac{{}_rN}{bD\cdot{}_mf_c}&=\frac{x_{n1}}{n(x_{n1}-{}_rd_{c1})}\;{}_rn\\ \frac{{}_rM}{bD^2\cdot{}_mf_c}&=\frac{x_{n1}}{n(x_{n1}-{}_rd_{c1})}\;{}_rm\end{aligned}\right\}\quad(26)$$

(iii) For the case controlled by reinforcing bars in tension.

(a) When $0\leqq x_{n1}\leqq 1$

$$\left.\begin{aligned} \frac{{}_rN}{bD\cdot{}_mf_t}&=\frac{x_{n1}}{n(1-{}_rd_{t1}-x_{n1})}\;{}_rn\\ \frac{{}_rM}{bD^2\cdot{}_mf_t}&=\frac{x_{n1}}{n(1-{}_rd_{t1}-x_{n1})}\;{}_rm\end{aligned}\right\}\quad(27)$$

(b) When $x_{n1}<0$

$$\left.\begin{aligned} \frac{{}_rN}{bD\cdot{}_mf_t}&=\frac{{}_mp_t(2x_{n1}-1)}{1-{}_rd_{t1}-x_{n1}}\\ \frac{{}_rM}{bD^2\cdot{}_mf_t}&=\frac{{}_mp_t(1-2{}_rd_{t1})^2}{1-{}_rd_{t1}-x_{n1}}\end{aligned}\right\}\quad(28)$$

(5) f_c' shall be as given by Eq. (29).

$$f_c'=f_c(1-15{}_sp_c)\qquad(\text{kg/cm}^2)\qquad(29)$$

2. In addition to Eqs. (10) through (15), calculation for columns subjected to combined axial force and bending moment may be as given by Eq. (30).

$$\left.\begin{aligned} N&={}_sN+{}_rN &&(\text{kg})\\ M&\leqq{}_sM+{}_rM &&(\text{kg}\cdot\text{cm})\end{aligned}\right\}\quad(30)$$

3. Calculation for columns with unsymmetrical cross section shall be as given by Eqs. (10) through (15) with replacing the section by conservative symmetrical cross section, or by Eq. (30) taking the unsymmetry into consideration.

4. Calculation for columns subjected to combined axial force and biaxial bending moments may be as given by Eq. (31).

$$\left.\begin{aligned} N&={}_sN+{}_rN &&(\text{kg})\\ M_x&\leqq{}_sM_x+{}_rM_x &&(\text{kg}\cdot\text{cm})\\ M_y&\leqq{}_sM_y+{}_rM_y &&(\text{kg}\cdot\text{cm})\end{aligned}\right\}\quad(31)$$

5. Calculation of columns for which effective buckling length exceeds 12 times the depth of the cross section shall be as given by Eqs. (32) through (35).

(1) Calculation shall be as given by equations in either (i) or (ii).

(i) (a) When $N\leqq{}_rN_c$ or $M\geqq{}_sM_0(1-{}_r\nu\cdot{}_rN_c/N_k)$

$$\left.\begin{aligned} N&={}_rN &&(\text{kg})\\ M&\leqq{}_sM_0\left(1-\frac{{}_r\nu\cdot{}_rN}{N_k}\right)+{}_rM &&(\text{kg}\cdot\text{cm})\end{aligned}\right\}\quad(32)$$

(b) When $N>{}_rN_c$ or $M<{}_sM_0(1-{}_r\nu\cdot{}_rN_c/N_k)$

$$\left.\begin{aligned} N&\leqq{}_rN_c+{}_sN &&(\text{kg})\\ M&={}_sM\left(1-\frac{{}_r\nu\cdot{}_rN_c}{N_k}\right) &&(\text{kg}\cdot\text{cm})\end{aligned}\right\}\quad(33)$$

(ii) (a) When $N\leqq{}_sN_c$ or $M\geqq{}_rM_0(1-{}_s\nu\cdot{}_sN_c/N_k)$

$$\left.\begin{aligned} N&={}_sN &&(\text{kg})\\ M&\leqq{}_rM_0\left(1-\frac{{}_s\nu\cdot{}_sN}{N_k}\right)+{}_sM &&(\text{kg}\cdot\text{cm})\end{aligned}\right\}\quad(34)$$

(b) When $N>{}_sN_c$ or $M<{}_rM_0(1-{}_s\nu\cdot{}_sN_c/N_k)$

$$\left.\begin{aligned} N&\leqq{}_sN_c+{}_rN &&(\text{kg})\\ M&={}_rM\left(1-\frac{{}_s\nu\cdot{}_sN_c}{N_k}\right) &&(\text{kg}\cdot\text{cm})\end{aligned}\right\}\quad(35)$$

where ${}_r\nu$ and ${}_s\nu$ shall be taken equal to 3.0 and 1.5, respectively, for the long-term loading condition, and equal to 1.5 and 1.0, respectively, for the short-term loading condition.

(2) N_k shall be as given by Eq. (36).

$$N_k=\frac{\pi^2}{l_k^2}\left(\frac{{}_cE\cdot{}_cI}{5}+{}_sE\cdot{}_sI\right)\qquad(\text{kg})\qquad(36)$$

(3) ${}_sN$ and ${}_sN_c$ are allowable compressive strengths of the steel portion as a slender column, and shall be as given according to Design Standards for Steel Structures.

(4) ${}_rN$ and ${}_rN_c$ are allowable compressive strengths of the reinforced concrete portion as a slender column, and shall be as given according to Item 1 (4) of this article for the cross section subjected to axial compression ${}_rN$ and bending moment equal to ${}_rM$ multiplied by ${}_r\delta$ given by Eq. (37). However, end eccentricities not less than 5% of the depth of concrete section shall be considered in the calculation.

$${}_r\delta=\frac{1}{1-\dfrac{{}_r\nu\cdot{}_rN}{{}_rN_k}}\qquad(37)$$

where ${}_rN_k$ shall be as given by Eq. (38).

$${}_rN_k=\frac{2\pi^2\cdot{}_cE\cdot{}_cI}{5l_k^2}\qquad(\text{kg})\qquad(38)$$

6. Compression force acting on a column under the earthquake loading shall be not larger than the value specified by Eq. (39).

$$N_l \leqq {}_r\mu \cdot bD \cdot F_c + {}_s\mu \cdot {}_sA \cdot {}_sf_c \qquad \text{(kg)} \qquad (39)$$

where ${}_r\mu$ = 1/3 and ${}_s\mu$ = 2/3 may be taken.

Article 18 Calculation for Shear and Torsional Strengths of Members

1. Calculation for shear and torsional strengths of beams and columns shall conform to this Article. Calculation for the bond of main reinforcing bars and steel shall conform to Article 19.

2. (1) Calculation for shear force in a beam shall be as given by Eqs. (40) and (41).

$$_sQ_d \leqq {}_sQ_A \qquad \text{(kg)} \qquad (40)$$

$$_rQ_d \leqq {}_rQ_A \qquad \text{(kg)} \qquad (41)$$

(2) Allowable shear strength of the steel portion ${}_sQ_A$ shall be as given by Eq. (42).

$$_sQ_A = \begin{cases} t_w \cdot d_w \cdot {}_sf_s & \text{: full-web type} \quad \text{(kg)} \\ {}_DA \cdot {}_sf_t \sin {}_D\theta & \text{: lattice type} \quad \text{(kg)} \\ \dfrac{2M_{BA}}{x_B} & \text{: batten-plate type} \quad \text{(kg)} \end{cases} \qquad (42)$$

(3) Allowable shear strength of the reinforced concrete portion ${}_rQ_A$ shall be as given by Eqs. (43) and (44).

$$_rQ_A = \min({}_rQ_{A1}, {}_rQ_{A2}) \qquad \text{(kg)} \qquad (43)$$

$$\left.\begin{aligned} {}_rQ_{A1} &= b \cdot {}_rj({}_r\alpha \cdot f_s + 0.5{}_wp \cdot {}_wf_t) \qquad \text{(kg)} \\ {}_rQ_{A2} &= b \cdot {}_rj\left(2\frac{b'}{b}f_s + {}_wp \cdot {}_wf_t\right) \qquad \text{(kg)} \end{aligned}\right\} \qquad (44)$$

where ${}_wp$ and ${}_r\alpha$ shall be as given by Eqs. (1) and (45), respectively. ${}_wp$ shall be taken equal to 0.6%, when it exceeds 0.6%.

$$_r\alpha = \frac{4}{\dfrac{{}_rM_d}{{}_rQ_d \cdot {}_rd} + 1} \quad \text{and} \quad 1 \leqq {}_r\alpha \leqq 2 \qquad (45)$$

(4) Design shear force for the steel and the reinforced concrete portions, ${}_sQ_d$ and ${}_rQ_d$, shall be as given by Eqs. (46) and (47).

$$_sQ_d = \frac{{}_sM_d}{M}Q \qquad \text{(kg)} \qquad (46)$$

$$_rQ_d = \frac{{}_rM_d}{M}Q \qquad \text{(kg)} \qquad (47)$$

However ${}_rQ_d$ shall be as given either by Eq. (48) or Eq. (49), when the lateral load resisting capacity is not examined.

$$_rQ_d = \frac{{}_rM_d}{M}Q_0 + \frac{{}_rM_1 + {}_rM_2}{l'} \qquad \text{(kg)} \qquad (48)$$

$$_rQ_d = \frac{{}_rM_d}{M}(Q_0 + 2Q_E) \qquad \text{(kg)} \qquad (49)$$

(5) When the design calculation is made according to Item 2 of Article 15 instead of the requirements in (1) through (4) described above, allowable shear strength shall be as given by Eqs. (50) and (51).

$$Q_A = \min(Q_{A1}, Q_{A2}) \qquad \text{(kg)} \qquad (50)$$

$$\left.\begin{aligned} Q_{A1} &= b \cdot {}_rj\{\alpha \cdot f_s + 0.5{}_wf_t({}_wp_e - 0.002)\} \qquad \text{(kg)} \\ Q_{A2} &= b \cdot {}_rj\left(2\frac{b'}{b}f_s + {}_wp_e \cdot {}_wf_t\right) \qquad \text{(kg)} \end{aligned}\right\} \qquad (51)$$

where ${}_wp_e$ and α shall be as given by Eqs. (52) and (53), respectively.

$$_wp_e = \frac{1}{2} \cdot \frac{T_B}{b \cdot x_B \cdot {}_wf_t} + {}_wp \qquad (52)$$

$$\alpha = \frac{4}{\dfrac{M}{Q \cdot {}_rd} + 1} \quad \text{and} \quad 1 \leqq \alpha \leqq 2 \qquad (53)$$

where ${}_wp_e$ shall be taken equal to 1.2%, when it exceeds 1.2%.

(6) For truss or lattice type steel members, the effect of eccentricities at web-to-chord connections shall be taken into account in the design calculation, if it is large.

3. (1) Calculation for shear force of columns shall be as given by Eqs. (54) through (56).

For long-term stress condition: $Q \leqq Q_A$ (kg) (54)

For short-term stress condition: ${}_sQ_d \leqq {}_sQ_A$ (kg) (55)

${}_rQ_d \leqq {}_rQ_{AS}$ (kg) (56)

(2) Allowable shear strength for the long-term loading condition Q_A shall be as given by Eq. (57)

$$Q_A = (1 + \beta){}_rQ_{AL} \qquad \text{(kg)} \qquad (57)$$

where ${}_rQ_{AL}$ shall be as given by Eq. (58).

$$_rQ_{AL} = b \cdot {}_rj \cdot \alpha' \cdot f_s \qquad \text{(kg)} \qquad (58)$$

where α, α' and β shall be as given by Eqs. (53), (59) and (60), respectively.

$$\alpha' = \begin{cases} \alpha & : b'/b \geqq \alpha/3 \\ 3\dfrac{b'}{b} & : b'/b < \alpha/3 \end{cases} \qquad (59)$$

$$\beta=\begin{cases}\dfrac{15t_w\cdot d_w}{b\cdot {}_rj} & \text{: full-web type and cruciform full-web type}\\ \dfrac{20b_f\cdot t_f}{b\cdot {}_rj} & \text{: full-web type bent about weak axis}\\ \dfrac{35{}_DA\cdot\cos{}_D\theta\cdot\sin^2{}_D\theta}{b\cdot {}_rj} & \text{: lattice type}\\ 0 & \text{: batten-plate type}\end{cases} \qquad (60)$$

(3) Allowable shear strength of the steel portion for the short-term loading condition ${}_sQ_A$ shall be as given by Eq. (42). However, for the cruciform full-web type steel or the full-web type steel subjected to weak-axis bending, it shall be as given by Eq. (61).

$${}_sQ_A=\begin{cases}d_w\cdot t_w\cdot {}_sf_s & \text{: cruciform full-web type} \quad (\text{kg})\\ \dfrac{4}{3}b_f\cdot t_f\cdot {}_sf_s & \text{: full-web type bent about weak axis} \quad (\text{kg})\end{cases} \qquad (61)$$

(4) Allowable shear strength of the reinforced concrete portion for the short-term loading condition ${}_rQ_{AS}$ shall be as given by Eqs. (62) and (63).

$${}_rQ_{AS}=\min({}_rQ_{AS1},\ {}_rQ_{AS2}) \quad (\text{kg}) \qquad (62)$$

$$\left.\begin{array}{ll}{}_rQ_{AS1}=b\cdot {}_rj(f_s+0.5{}_wp\cdot {}_wf_t) & (\text{kg})\\ {}_rQ_{AS2}=b\cdot {}_rj\left(2\dfrac{b'}{b}f_s+{}_wp\cdot {}_wf_t\right) & (\text{kg})\end{array}\right\} \qquad (63)$$

where ${}_wp$ shall be as given by Eq. (1). However, ${}_wp$ shall be taken equal to 0.6%, if it exceeds 0.6%.

(5) Design shear force of the steel and the reinforced concrete portions for the short-term loading condition ${}_sQ_d$ and ${}_rQ_d$ shall be as given by Eqs. (46) and (47). However, ${}_rQ_d$ shall be as given either by Eq. (64) or Eq. (49), when the lateral load resisting capacity is not examined.

$${}_rQ_d=\frac{{}_rM_1+{}_rM_2}{h'} \quad (\text{kg}) \qquad (64)$$

where ${}_rM_1$ and ${}_rM_2$ shall be as given for a column subjected to the compression specified in Table 14.

Table 14 Compression Force for the Computation of Ultimate Flexural Strength (kg)

Range	Compression force
$N\leqq {}_rN_c$	N
$N> {}_rN_c$	${}_rN_c$

When the remaining bending moment after subtracting the allowable flexural strength for the short-term loading condition of the steel portion at the column top from a half the sum of the ultimate bending moments of two adjacent beams is less

than the ultimate bending moment of the reinforced concrete portion, the latter may be taken as the remaining bending moment. For the column top of the top story, "a half" shall be omitted.

(6) When the design calculation is made according to Item 2 of Article 15, allowable shear strengths shall be as given by Eq. (57) for the long-term loading condition, and by Eqs. (65) and (66) for the short-term loading condition, respectively.

$$Q_{AS}=\min(Q_{AS1},\ Q_{AS2}) \quad (\text{kg}) \qquad (65)$$

$$\left.\begin{array}{ll}Q_{AS1}=b\cdot {}_rj\{f_s+0.5{}_wf_t({}_wp_e-0.002)\} & (\text{kg})\\ Q_{AS2}=b\cdot {}_rj\left(2\dfrac{b'}{b}f_s+{}_wp_e\cdot {}_wf_t\right) & (\text{kg})\end{array}\right\} \qquad (66)$$

where ${}_wp_e$ shall be as given by Eq. (52).

(7) For truss or lattice type steel members, the effect of eccentricities at web-to-chord connections shall be taken into account in the calculation, if it is large.

4. (1) Calculation for shear strength of beams with openings shall be as given by Eqs. (40) and (41).

(2) Allowable shear strength of the steel portion with an opening in the web ${}_sQ_A$ shall be as given by Eq. (67).

$${}_sQ_A=\gamma_h\cdot t_w(d_w-D_h){}_sf_s \quad (\text{kg}) \qquad (67)$$

where γ_h shall be taken as the values specified in Eq. (68).

$$\gamma_h=\begin{cases}1.0 & \text{: for openings with flanges}\\ 0.85 & \text{: for openings without flanges}\end{cases} \qquad (68)$$

(3) Allowable shear strength of the reinforced concrete portion with an opening ${}_rQ_A$ shall be as given by Eqs. (69) and (70).

$${}_rQ_A=\min({}_rQ_{A1},\ {}_rQ_{A2}) \quad (\text{kg}) \qquad (69)$$

$$\left.\begin{array}{ll}{}_rQ_{A1}=b\cdot {}_rj\left\{{}_r\alpha\cdot f_s\left(1-1.6\dfrac{D_h}{D}\right)+0.5{}_wp_h\cdot {}_wf_t\right\} & (\text{kg})\\ {}_rQ_{A2}=b\cdot {}_rj\left(2\dfrac{b'}{b}f_s+{}_wp_h\cdot {}_wf_t\right) & (\text{kg})\end{array}\right\} \qquad (70)$$

where ${}_wp_h$ shall be as given by Eq. (71).

$${}_wp_h=\frac{\Sigma\{{}_wa(\sin{}_w\theta+\cos{}_w\theta)\}}{bD} \qquad (71)$$

${}_wp_h$ shall be as given for web reinforcement placed within a distance of half the beam depth apart from the center of the opening on either side of the opening.

5. Allowable torsional strength of a member may be taken as the sum of allowable tortional strength of the reinforced concrete and the steel portions.

Article 19 Bond for Reinforcing Bars and Steel

1. (1) Calculation of bond for tensile reinforcing bars in flexral members shall be as given by Eq. (72) or (73).

$$ {}_m\tau_a = \frac{{}_rQ_d}{{}_m\psi \cdot {}_rj} \leqq {}_mf_a \qquad (\mathrm{kg/cm^2}) \qquad (72)$$

$$ l_d \geqq \frac{{}_ma_t \cdot {}_m\sigma_d}{0.8{}_mf_a \cdot {}_m\psi} + {}_rj \qquad (\mathrm{cm}) \qquad (73)$$

(2) Design shear force for the reinforced concrete portion ${}_rQ_d$ shall conform to Article 18. However, for a column under the long-term loading condition, the design shear force shall be as given by Eq. (74).

$$ {}_rQ_d = \frac{Q}{1+\beta} \qquad (\mathrm{kg}) \qquad (74)$$

where β is given by Eq. (60).

2. When the design is made according to Item 2 of Article 15, the calculation for bond along longitudinal steel and reinforcing bars shall be as given on the assumption that the design shear force is distributed to the longitudinal steel in tension and the longitudinal reinforcing bars in tension in proportion to their cross-sectional areas.

Article 20 Beam-to-Column Connections

1. Beam-to-column connections shall be designed to transfer axial force, bending moment and shear force acting at the ends of columns and beams.

2. Design calculation for shear force of connection panels surrounded by columns and beams shall be as given by Eqs. (75) and (76).

(1) Calculation for the long-term loading condition shall be as given by Eq. (75).

$$ {}_cV \cdot 3f_s(1+\beta) \geqq \frac{h'}{h}({}_BM_1 + {}_BM_2) \qquad (\mathrm{kg \cdot cm}) \qquad (75)$$

(2) Calculation for the short-term stress condition shall be made by Eq. (76).

$$ {}_cV_e(2f_s \cdot {}_J\delta + {}_wp \cdot {}_wf_t) + {}_sV \cdot {}_sf_s \geqq \frac{h'}{h}({}_BM_1 + {}_BM_2) \qquad (\mathrm{kg \cdot cm}) \qquad (76)$$

(3) The values of ${}_cV$, ${}_cV_e$, ${}_sV$ and β shall be as given according to expression in Table 15, and ${}_J\delta$ is given in Table 16.

Table 15 Values of ${}_cV$, ${}_cV_e$, ${}_sV$ and β

	${}_cV$	${}_cV_e$	${}_sV$	β
With SRC or RC beams	${}_cb \cdot {}_{mB}d \cdot {}_{mC}d$	$\frac{{}_cb + {}_Bb}{2}{}_{mB}d \cdot {}_{mC}d$	${}_Jt_w \cdot {}_{sB}d \cdot {}_{sC}d$	$\frac{15{}_Jt_w \cdot {}_{sC}d}{{}_cb \cdot {}_{mC}d}$
With S beams	${}_cb \cdot {}_{sB}d \cdot {}_{mC}d$	$\frac{{}_cb}{2}{}_{sB}d \cdot {}_{mC}d$		

Table 16 Value of ${}_J\delta$

Shape of connection	${}_J\delta$
Cross-shape	3
T-shape	2
L-shape	1

3. When the sum of flexural strengths of the steel portions or the reinforced concrete portions of columns and beams adjacent to a connection satisfies either Eq. (77) or (78), the stress transfer between columns and beams through the connection may not be examined.

$$ 0.4 \leqq \frac{{}_{sC}M_A}{{}_{sB}M_A} \leqq 2.5 \qquad (77)$$

$$ 0.4 \leqq \frac{{}_{rC}M_A}{{}_{rB}M_A} \qquad (78)$$

4. Cross-sectional area of a flange stiffener in the connection shall be as given by Eq. (79).

$$ A_s = \frac{P - {}_sf_c' \cdot {}_ct_w({}_Bt_f + 5d_f)}{{}_sf_t} \qquad (\mathrm{cm^2}) \qquad (79)$$

where ${}_sf_c' = F/1.3$ for the long-term loading condition.

5. When openings for reinforcing bars are placed in the steel portion, proper design shall be made to ensure that the deformation capacity of the member should not be reduced.

6. When base plates and anchor bolts are used at a connection between different types of members, the design shall conform to Article 22.

Article 21 Joints

1. (1) Steel joints of beams and columns shall be designed for axial force, bending moment and shear force specified by Eq. (80).

$$\left.\begin{aligned} {}_sN_j &= {}_sN && \text{(kg)} \\ {}_sM_j &= \frac{{}_sM_d}{M} M_j && \text{(kg·cm)} \\ {}_sQ_j &= \frac{{}_sM_d}{M} Q_j && \text{(kg)} \end{aligned}\right\} \quad (80)$$

where ${}_sM_d/M$ shall be taken as the value used in the calculation at the member end. However, the design stresses for the short-term loading condition shall conform to (2) of this Item, when the lateral load resisting capacity is not examined.

(2) Design axial force shall be as given by Eq. (80), and design bending moment and shear force shall be as given by Eq. (81), (82) or (83) for the short-term loading condition, when the lateral load resisting capacity is not examined.

(i) ${}_sM_j$ and ${}_sQ_j$ for beam joints shall be as given by Eq. (81).

$$\left.\begin{aligned} {}_sM_j &= Y\left[\frac{{}_sM_d}{M} M_{j0} + \nu_j\left\{{}_sM_1 - \frac{l_j}{l'}({}_sM_1 + {}_sM_2)\right\}\right] && \text{(kg·cm)} \\ &\quad \text{but not larger than } Y\cdot\nu_j\cdot{}_sM_A \\ {}_sQ_j &= Y\left\{\frac{{}_sM_d}{M} Q_{j0} + \frac{\nu_j}{l'}({}_sM_1 + {}_sM_2)\right\} && \text{(kg)} \\ &\quad \text{but not larger than } Y\cdot\nu_j\cdot{}_sQ_A \end{aligned}\right\} \quad (81)$$

(ii) ${}_sM_j$ and ${}_sQ_j$ for column joints shall be as given by Eq. (82).

$$\left.\begin{aligned} {}_sM_j &= Y\cdot\nu_j\left\{{}_sM_1 - \frac{h_j}{h'}({}_sM_1 + {}_sM_2)\right\} && \text{(kg·cm)} \\ {}_sQ_j &= \frac{Y\cdot\nu_j}{h'}({}_sM_1 + {}_sM_2) \quad \text{but not larger than } Y\cdot\nu_j\cdot{}_sQ_A && \text{(kg)} \end{aligned}\right\} \quad (82)$$

where ${}_sM_1$ and ${}_sM_2$ shall be as given for the column subjected to ${}_sN$. When the remaining bending moment after subtracting the ultimate bending moment of the reinforced concrete portion at the column top from a half the sum of the ultimate bending moment of two adjacent beams is less than the allowable flexural strength for the short-term loading condition of the steel portion, ${}_sM_1$ or ${}_sM_2$ may be taken as the remaining bending moment. For the column top of the top story, "a half" shall be omitted. The ultimate bending moment of the reinforced concrete portion at the column top shall be as given for the column subjected to $(N\text{-}{}_sN)$.

(iii) ${}_sM_j$ and ${}_sQ_j$ of a beam or a column may be as given by Eq. (83), instead of Eq. (81) or (82).

$$\left.\begin{aligned} {}_sM_j &= \frac{{}_sM_d}{M}(M_{Lj} + 2M_{Ej}) \quad \text{but not larger than } Y\cdot\nu_j\cdot{}_sM_A && \text{(kg·cm)} \\ {}_sQ_j &= \frac{{}_sM_d}{M}(Q_{Lj} + 2Q_{Ej}) \quad \text{but not larger than } Y\cdot\nu_j\cdot{}_sQ_A && \text{(kg)} \end{aligned}\right\} \quad (83)$$

where Y and ν_j shall conform to Table 17.

Table 17 Values of Y and ν_j

	SS400, STK400, STKR400 SM400, SMA400	SS490, SM490, STK490 STKR490, SCW490-CF	SM490Y SMA490	SM520 SCW520-CF
Y	0.59	0.66	0.74	0.70
ν_j	1.2	1.1	1.1	1.1

(3) Allowable bending moment and allowable shear force of steel joints shall conform to Design Standards for Steel Structures.

(4) Reinforced concrete portion at the steel joint shall be safely designed for design axial force, bending moment and shear force acting at the joint subtracted by ${}_sN_j$, ${}_sM_j$ and ${}_sQ_j$ specified by Eq. (80), respectively. However, allowable shear force for the short-term loading condition shall be as given by Eq. (48) or (49) for a beam, and by Eq. (64) for a column, when the lateral load resisting capacity is not examined.

2. When allowable bending moment or shear force for the short-term loading condition of the steel joint is less than ${}_sM_j$ or ${}_sQ_j$ specified by Eqs. (81), (82) or (83), the remaining design force may be carried by the reinforced concrete portion, provided that stress transfer can be assured near the joint. However, allowable bending moment and shear force for the short-term loading condition of the steel joint shall be not less than a half the respective strength of the base steel.

3. Design for the joints of reinforcing bars shall conform to Standards for Structural Calculation of Reinforced Concrete Structures.

Article 22 Column Bases

1. Column bases shall be designed for axial force, bending moment and shear force.

2. (1) Design calculation for axial force and bending moment of bare type of column bases shall be as given according to Article 17, Item 1, (1) or Item 2, where ${}_sN$ and ${}_sM$ shall be taken as allowable axial and bending strength of the cross section composed of the anchor bolts and the concrete underneath the base plate, respectively, and ${}_rN$ and ${}_rM$ as allowable axial and bending strength of the reinforced concrete portion surrounding the column base, respectively.

(2) ${}_sN$ and ${}_sM$ shall be as given by Eqs. (84) through (86).

(i) When $\frac{{}_aN_t}{2} < {}_sN \leq {}_bN_c + \frac{{}_aN_t}{2}$

$${}_sN = {}_bN + \frac{{}_aN_t}{2} \quad \text{(kg)}$$
$${}_sM = {}_bM + {}_aM_0 \quad \text{(kg·cm)} \qquad (84)$$

(ii) When ${}_bN_c + \frac{{}_aN_t}{2} < {}_sN \leq {}_bN_c$

$${}_sN = {}_bN_c + {}_aN \quad \text{(kg)}$$
$${}_sM = {}_aM \quad \text{(kg·cm)} \qquad (85)$$

(iii) When ${}_aN_t \leq {}_sN \leq \frac{{}_aN_t}{2}$

$${}_sN = {}_aN \quad \text{(kg)}$$
$${}_sM = {}_aM \quad \text{(kg·cm)} \qquad (86)$$

(3) ${}_rN$ and ${}_rM$ shall be as given by Eqs. (87) through (89).

(i) When $0 \leq {}_rN \leq {}_cN_c$

$${}_rN = {}_cN \quad \text{(kg)}$$
$${}_rM = {}_cM + {}_mM_0 \quad \text{(kg·cm)} \qquad (87)$$

(ii) When ${}_cN_c < {}_rN \leq {}_cN_c + {}_mN_c$

$${}_rN = {}_cN_c + {}_mN \quad \text{(kg)}$$
$${}_rM = {}_mM \quad \text{(kg·cm)} \qquad (88)$$

(iii) When ${}_mN_t \leq {}_rN < 0$

$${}_rN = {}_mN \quad \text{(kg)}$$
$${}_rM = {}_mM \quad \text{(kg·cm)} \qquad (89)$$

(4) ${}_bN$ and ${}_bM$ shall be as given according to Table A1 in Appendix, ${}_aN$ and ${}_aM$ to Table A2, and ${}_bN_c$, ${}_aN_t$ and ${}_aM_0$ to Table A3.
(5) ${}_cN$ and ${}_cM$ shall conform to Table A4 in Appendix, ${}_mN$ and ${}_mM$ to Table A5, and ${}_cN_c$, ${}_mN_c$, ${}_mN_t$, and ${}_mM_0$ to Table A6. f_c' for the computation of allowable strength may be taken as the values used for the calculation of the column.
(6) Base plate and anchor bolts shall be safely designed for stresses acting on them.

3. (1) Design bending moment at the bottom of base plate of embedded type of column bases shall be as given by Eq. (90).

$$M = {}_sM_a + \frac{{}_sQ_a \cdot {}_bh}{2} - \frac{b_e \cdot f_B}{4}\left\{{}_bh^2 - \left(\frac{{}_sQ_a}{b_e \cdot f_B}\right)^2\right\} \quad \text{(kg·cm)} \qquad (90)$$

(2) f_B shall be as given by Eq. (91).

$$f_B = \min\left(\sqrt{\frac{b}{b_e}}f_c,\ 12f_c,\ \frac{{}_wa \cdot {}_wf_t}{b_e \cdot x}\right) \quad \text{(kg/cm}^2\text{)} \qquad (91)$$

However, term ${}_wa \cdot {}_wf_t / b_e \cdot x$ may not be considered for inner column bases.
(3) b_e shall conform to Table 18.

Table 18 Effective Width of Embedded Type of Column Base (cm)

Shape and direction	b_e
H-shape bent about strong axis	$t_w + 2d_f$
H-shape bent about weak axis	$2t_w + 2d_f$
Cross-shape	$3t_w + 4d_f$

4. Allowable shear strength of bare type of column bases shall be taken as the sum of allowable shear strength underneath the base plate and of the reinforced concrete portion surrounding the column base.

Article 23 Footings

Design calculation for reinforced concrete footings shall conform to Standards for Structural Calculation of Reinforced Concrete Structures.

Article 24 Slabs

1. Design calculation for reinforced concrete slabs shall conform to Standards for Structural Calculation of Reinforced Concrete Structures.

2. Design calculation of composite slabs considering composite effects of deck plates shall conform to Design Recommendations for Composite Construction, Part 2: Design Recommendations for Composite Slabs Made of Deck Plates and Concrete.

Article 25 Structural Walls

1. Columns and footings adjacent to structural walls shall be designed for vertical load, over-turning moment and axial force specified in Item 4 of this article, and structural walls shall be designed for shear force.

2. (1) Allowable shear strength for the short-term loading condition per one span of a structural wall subjected to horizontal forces shall be as given by Eqs. (92) through (94).

$${}_wQ_A = \max({}_wQ_{A1},\ {}_wQ_{A2}) \quad \text{(kg)} \qquad (92)$$
$${}_wQ_{A1} = r \cdot {}_wt \cdot l \cdot f_s(1+\beta) \quad \text{(kg)} \qquad (93)$$

$${}_wQ_{A2}=\max({}_wQ'_{A1},\ {}_wQ'_{A2})+{}_sQ_A \qquad \text{(kg)} \qquad (94)$$

(2) ${}_wQ'_{A1}$ and ${}_wQ'_{A2}$ shall be as given by Eqs. (95) and (97).

$${}_wQ'_{A1}=r\cdot\min\{f'_s,\ ({}_wp\cdot{}_wf_t+{}_w\tau_A)\}\,{}_wt\cdot l'+\frac{1}{2}\min\left(\Sigma_{CS}Q_A,\ \Sigma_{BS}Q_A\frac{l}{h}\right) \qquad \text{(kg)} \qquad (95)$$

where f'_s shall be as given by Eq. (96).

$$f'_s=\min(0.3f_c,\ 3.51\sqrt{f_c}) \qquad (\text{kg/cm}^2) \qquad (96)$$

However, ${}_{CS}Q_A$ and ${}_{BS}Q_A$ shall be taken as 0, in general, if the wall under consideration is not neighbored by other walls at either side.

$${}_wQ'_{A2}=\min\left(\Sigma_{CT}R_A,\ \Sigma_{BT}R_A\frac{l}{h}\right) \qquad \text{(kg)} \qquad (97)$$

However, ${}_{CT}R_A$ and ${}_{BT}R_A$ shall be taken as 0, in general, if the wall under consideration is neighbored by other walls at either side.

(3) β and ${}_sQ_A$ shall be as given by Eqs. (98) and (99), respectively.

$$\beta=\begin{cases}\dfrac{35\,{}_DA\cdot\cos{}_D\theta\cdot\sin{}_D\theta}{{}_wt\cdot l} & \text{: containing steel braces of flat bars or shapes}\\[2ex] 15\dfrac{{}_swt}{{}_wt} & \text{: containing steel plates}\end{cases} \qquad (98)$$

$${}_sQ_A=\begin{cases}{}_DA\cdot{}_sf_t\cdot\cos{}_D\theta & \text{: containing steel braces of flat bars or shapes} \quad \text{(kg)}\\ {}_sf_s\cdot{}_swt\cdot l' & \text{: containing steel plates} \quad \text{(kg)}\end{cases} \qquad (99)$$

(4) ${}_w\tau_A$ shall be as given by Eq. (100).

$${}_w\tau_A=\min\left(\frac{{}_{BS}\beta_b}{{}_wt\cdot l^2}{}_{BS}M_A,\ \frac{{}_{CS}\beta_b}{{}_wt\cdot h^2}{}_{CS}M_A,\ \frac{{}_{BS}\beta_S}{{}_wt\cdot l}{}_{BS}Q_A,\ \frac{{}_{CS}\beta_S}{{}_wt\cdot h}{}_{CS}Q_A\right) \qquad (\text{kg/cm}^2) \qquad (100)$$

where ${}_{BS}\beta_b$, ${}_{CS}\beta_b$, ${}_{BS}\beta_s$ and ${}_{CS}\beta_s$ are constants related to the stress condition around the wall. ${}_{BS}M_A$, ${}_{CS}M_A$, ${}_{BS}Q_A$ and ${}_{CS}Q_A$, are allowable flexural and shear strengths of beams and columns surrounding the wall, respectively, and shall be taken as the smaller of those of the upper and lower beams, and those of the columns on the right and left hand sides, respectively. When the wall under consideration is neighbored by other walls at both sides, $\frac{{}_{BS}\beta_b}{{}_wt\cdot l^2}{}_{BS}M_A$ and $\frac{{}_{BS}\beta_s}{{}_wt\cdot l}{}_{BS}Q_A$ for beams, and $\frac{{}_{CS}\beta_b}{{}_wt\cdot h^2}{}_{CS}M_A$ and $\frac{{}_{CS}\beta_s}{{}_wt\cdot h}{}_{CS}Q_A$,for columns need not be considered.

(5) ${}_{CT}R_A$ and ${}_{BT}R_A$ shall be as given by Eqs. (101) and (102).

$${}_{CT}R_A=\min\left(\frac{2\,{}_{CT}M_A}{h_h},\ {}_{CT}Q_A\right) \qquad \text{(kg)} \qquad (101)$$

$${}_{BT}R_A=\min\left(\frac{2\,{}_{BT}M_A}{l_h},\ {}_{BT}Q_A\right) \qquad \text{(kg)} \qquad (102)$$

(6) r shall be as given by Eq. (103).

$$r=\min(r_1,\ r_2,\ r_3) \qquad (103)$$

where r_1, r_2, and r_3 shall be as given by Eq. (104).

$$\left.\begin{aligned} r_1&=1-\frac{l_h}{l}\\ r_2&=1-\sqrt{\frac{h_h\cdot l_h}{h\cdot l}}\quad\left(\text{but}\ \sqrt{\frac{h_h\cdot l_h}{h\cdot l}}\leqq 0.4\right)\\ r_3&=1-\frac{h_h}{h}\end{aligned}\right\} \qquad (104)$$

3. Design shear force for the short-term loading condition of a structural wall shall be taken as the earthquake shear force Q_E given by Article 9, Item 1. However, Q_E multiplied by not less than 1.5 shall be used for the design, if the lateral load resisting capacity is not examined.

4. Calculation for beams and columns adjacent to structural walls shall be as given by considering tensile forces for beams and columns specified by Eqs. (105) and (106), respectively. However, h' shall be replaced by $h'/2$ for beams at the roof level, and l' by $l'/2$ for outer columns.

$${}_{BS}T=r\cdot\min\{(f'_s-{}_wp\cdot{}_wf_t),\ {}_w\tau_A\}\,{}_wt\cdot h' \qquad \text{(kg)} \qquad (105)$$

$${}_{CS}T=r\cdot\min\{(f'_s-{}_wp\cdot{}_wf_t),\ {}_w\tau_A\}\cdot{}_wt\cdot l' \qquad \text{(kg)} \qquad (106)$$

5. Openings in reinforced concrete walls shall be reinforced for the edge tension T_h specified by Eq. (107).

$$T_h=\begin{cases}\dfrac{h_h}{2(l-l_h)}Q & \text{: vertical direction} \quad \text{(kg)}\\[2ex] \dfrac{l_h}{2(h-h_h)}\cdot\dfrac{h}{l}Q & \text{: horizontal direction} \quad \text{(kg)}\end{cases} \qquad (107)$$

6. Connection between a structural wall and surrounding frame members shall be so designed that design shear force be transferred. When the shear transfer is not sufficient, the effect shall be considered in the design.

Chapter 5 Calculation of Lateral Load Resisting Capacity

Section 1 General

Article 26 Scope

Requirements in this chapter apply to the calculation of lateral load resisting capacity of each story. Design calculation of ultimate strengths of members and connections in Section 3 does not include that of composite steel tube and concrete structures, design calculation of which shall conform to Chapter 6.

Section 2 Calculation of Required Lateral Load Resisting Capacity

Article 27 Required Lateral Load Resisting Capacity

Calculation of the required lateral load resisting capacity shall, in general, conform to Building Standard Law Enforcement Order and Ministry of Construction Notification.

Article 28 Shape Characteristic Factor

Shape characteristic factor shall, in general, conform to Building Standard Law Enforcement Order and Ministry of Construction Notification.

Article 29 Structural Characteristic Factor

Structural characteristic factor shall, in general, conform to Building Standard Law Enforcement Order and Ministry of Construction Notification.

Section 3 Calculation of Ultimate Strength of Members and Connections

Article 30 General Assumptions

Ultimate strengths of members and connections shall be taken as the sum of ultimate strengths of the steel and the reinforced concrete portions.

Article 31 Material Strengths

Material strengths shall, in genersl, conform to Building Standard Law Enforcement Order and Ministry of Construction Notification. However, material strengths of centrifugal casting steel tubes for welded structure, SCW490-CF and SCW520-CF, shall be taken equivalent to those of SM490 and SM520, respectively.

Article 32 Ultimate Flexural Strength of Members

1.(1) Ultimate flexural strength of a member may be as given by Eqs. (108) through (110).

(i) When ${}_rN_{tU} \leqq N_U \leqq {}_rN_{cU}$

$$\left.\begin{array}{ll} N_U = {}_rN_U & \text{(kg)} \\ M_U = {}_sM_{U0} + {}_rN_U & \text{(kg·cm)} \end{array}\right\} \qquad (108)$$

(ii) When $N_U > {}_rN_{cU}$

$$\left.\begin{array}{ll} N_U = {}_rN_{cU} + {}_sN_U & \text{(kg)} \\ M_U = {}_sM_U & \text{(kg·cm)} \end{array}\right\} \qquad (109)$$

(iii) When $N_U < {}_rN_{tU}$

$$\left.\begin{array}{ll} N_U = {}_rN_{tU} + {}_sN_U & \text{(kg)} \\ M_U = {}_sM_U & \text{(kg·cm)} \end{array}\right\} \qquad (110)$$

(2) Ultimate flexural strength of the reinforced concrete portion may be as given by Eq. (111) through (113).

(i) When $0 \leqq {}_rN_U \leqq {}_cN_{cU}$

$$\left.\begin{array}{ll} {}_rN_U = {}_cN_U & \text{(kg)} \\ {}_rM_U = {}_cM_U + {}_mM_{U0} & \text{(kg·cm)} \end{array}\right\} \qquad (111)$$

(ii) When ${}_rN_U > {}_cN_{cU}$

$$\left.\begin{array}{ll} {}_rN_U = {}_cN_{cU} + {}_mN_U & \text{(kg)} \\ {}_rM_U = {}_mM_U & \text{(kg·cm)} \end{array}\right\} \qquad (112)$$

(iii) When ${}_rN_U < 0$

$$\left.\begin{array}{ll} {}_rN_U = {}_mN_U & \text{(kg)} \\ {}_rM_U = {}_mM_U & \text{(kg·cm)} \end{array}\right\} \qquad (113)$$

(3) ${}_cM_U$ shall be as given according to Table Bl in Appendix, where ${}_c\gamma_U$ shall be as given by Eq. (114).

$${}_c\gamma_U = 0.85 - 2.5\,{}_sp_c \qquad (114)$$

(4) ${}_mM_U$ shall be as given according to Table B2 in Appendix.

(5) ${}_sM_U$ shall be as given according to Table B3 in Appendix.

2. Ultimate flexural strength of a member may be as given by Eq. (115) or (116)

instead of Eqs. (108) through (110).

$$\left.\begin{array}{ll} N_U={}_cN_U+{}_mN_U+{}_sN_U & \text{(kg)} \\ M_U={}_cM_U+{}_mM_U+{}_sM_U & \text{(kg·cm)} \end{array}\right\} \quad (115)$$

$$\left.\begin{array}{ll} N_U={}_rN_U+{}_sN_U & \text{(kg)} \\ M_U={}_rM_U+{}_sM_U & \text{(kg·cm)} \end{array}\right\} \quad (116)$$

3. ${}_rM_U$ of a rectangular reinforced concrete section with symmetrically placed bars may be as given by the equations in Table B4 in Appendix, instead of Eqs. (111) through (113).

4. M_U of a rectangular reinforced concrete section with symmetrically placed bars and a full-web steel may be as given by equations in Table B5 in Appendix, instead of Eqs. (108) through (110).

5. Ultimate flexural strength of a main girder may be as given by Eq. (117) or (118).

$$M_U={}_sZ_p\cdot{}_s\sigma_Y+{}_ma_t\cdot{}_m\sigma_Y\cdot{}_md \qquad \text{(kg·cm)} \qquad (117)$$

$$M_U={}_sZ_p\cdot{}_s\sigma_Y+\frac{{}_sa_w}{8}{}_s\sigma_Y\cdot{}_md+\left(\frac{3}{4}{}_ma_t+\frac{1}{4}{}_ma_c\right){}_m\sigma_Y\cdot{}_md \qquad \text{(kg·cm)} \qquad (118)$$

Article 33 Ultimate Shear Strength of Members

1. Ultimate shear strength of a member shall be as given by Eq. (119).

$$Q_U={}_rQ_U+{}_sQ_U \qquad \text{(kg)} \qquad (119)$$

2. (1) Ultimate shear strength of the reinforced concrete portion shall be as given by Eqs. (120) through (122).

$${}_rQ_U=\min({}_rQ_{sU},\ {}_rQ_{bU}) \qquad \text{(kg)} \qquad (120)$$

$${}_rQ_{bU}=\Sigma\frac{{}_rM_U}{l'} \qquad \text{(kg)} \qquad (121)$$

$${}_rQ_{sU}=\min({}_rQ_{sU1},\ {}_rQ_{sU2}) \qquad \text{(kg)} \qquad (122)$$

(2) ${}_rM_U$ may be as given by equations in Table B4 in Appendix, where the compression force may be taken as the value in Table 14.

(3) ${}_rQ_{sU1}$ and ${}_rQ_{sU2}$ shall be as given by Eq. (123).

$$\left.\begin{array}{ll} {}_rQ_{sU1}=b\cdot{}_rj(0.5F_s\cdot{}_r\alpha+0.5{}_wp\cdot{}_w\sigma_Y) & \text{(kg)} \\ {}_rQ_{sU2}=b\cdot{}_rj\left(F_s\frac{b'}{b}+{}_wp\cdot{}_w\sigma_Y\right) & \text{(kg)} \end{array}\right\} \quad (123)$$

where ${}_wp$ and ${}_r\alpha$ shall be as given by Eqs. (1) and (45), respectively.

(4) F_s shall be as given by Eq. (124).

$$F_s=\min\left(0.15F_c,\ 22.5+\frac{4.5F_c}{100}\right) \qquad \text{(kg·cm}^2\text{)} \qquad (124)$$

3. (1) Ultimate shear strength of the steel portion shall be as given by Eqs. (125) and (126).

$${}_sQ_U=\min({}_sQ_{sU},\ {}_sQ_{bU}) \qquad \text{(kg)} \qquad (125)$$

$${}_sQ_{bU}=\Sigma\frac{{}_sM_U}{l'} \qquad \text{(kg)} \qquad (126)$$

(2) ${}_sM_U$ may be as given by equations in Table B3 in Appendix.

(3) ${}_sQ_{sU}$ shall be as given by Eq. (127).

$${}_sQ_{sU}=\begin{cases} t_w\cdot d_w\frac{{}_s\sigma_Y}{\sqrt{3}} & \text{: full-web and cruciform steels (kg)} \\ {}_DA\cdot{}_s\sigma_Y\cdot\sin{}_D\theta & \text{: lattice type steel (kg)} \\ \frac{2M_{BU}}{x_B} & \text{: batten-plate type steel (kg)} \\ \frac{4}{3}b_f\cdot t_f\frac{{}_s\sigma_Y}{\sqrt{3}} & \text{: full-web steel bent about weak-axis (kg)} \end{cases} \qquad (127)$$

Article 34 Ultimate Shear Strength of Beam-to-Column Connections

Ultimate shear strength of a beam-to-column connection shall be as given by Eq. (128).

$${}_jM_U={}_cV_e({}_jF_s\cdot{}_j\delta+{}_wp\cdot{}_w\sigma_Y)+\frac{1.2{}_sV\cdot{}_s\sigma_Y}{\sqrt{3}} \qquad \text{(kg·cm)} \qquad (128)$$

where ${}_cV_e$ and ${}_sV$ shall be as given by equations in Table 15, ${}_j\delta$ in Table 16, and ${}_jF_s$ by Eq. (129).

$${}_jF_s=\min\left(0.12F_c,\ 18+\frac{3.6F_c}{100}\right) \qquad \text{(kg/cm}^2\text{)} \qquad (129)$$

Article 35 Ultimate Strength of Joints

1. Ultimate flexural strength of a member at the steel joint subjected to axial force and bending moment shall be as given by Eqs. (108) through (110) or (116), where ${}_sM_U$ shall be taken as ultimate flexural strength of the steel joint based on the rupture strength of the steel material and ${}_rM_U$ as ultimate flexural strength of the reinforced concrete portion given by Eqs. (111) through (113), or instead by Eq. (115) with ${}_sM_U$ defined above and ${}_cM_U$ and ${}_mM_U$, ultimate strengths of the concrete and the main reinforcing bars, which shall be as given according to Article 32, Item 1 (3) and (4), respectively.

2. Ultimate shear strength of a member at the steel joint shall be as given by Eq. (119), where ${}_sQ_U$ shall be taken as ultimate shear strength of the steel joint based on the rupture strength of the steel material, and ${}_rQ_U$ as ultimate shear strength of the reinforced concrete portion as given by Eqs. (120) through (122).

Article 36 Ultimate Strength of Column Bases

1. (1) Ultimate flexural strength of a bare type of column base shall be as given according to Article 32, Item 1 (1) or Item 2. ${}_sN_U$ and ${}_sM_U$ shall be taken as ultimate axial and flexural strengths, respectlvely, of the cross section composed of the anchor bolts and the concrete underneath the base plate. ${}_rN_U$ and ${}_rM_U$ shall be taken as ultimate axial and flexural strengths, respectively, of the reinforced concrete portion surrounding the column base.

(2) ${}_sN_U$ and ${}_sM_U$ shall be taken as ultimate axial and flexural strengths of the reinforced concrete section, as given by treating anchor bolts as tensile reinforcing bars with ${}_cr_U = 0.85$.

(3) ${}_rN_U$ and ${}_rM_U$ shall be as given by Eqs. (111) through (113).

(4) ${}_cM_U$ and ${}_mM_U$ shall be as given according to Tables El and B2 in Appendix, respectively, where ${}_cr_U$ shall be as given by Eq. (114), and ${}_sp_c$ shall be taken as the value used for the calculation of the column.

2. (1) Ultimate flexural strength of an embedded type of column base shall be as given by Eqs. (108) through (111), or instead by Eq. (115) or (116), where ${}_sM_U$ shall be as given by Eq. (130).

$$ {}_sM_U=\min({}_sM_{U1},\ {}_sM_{U2}) \qquad \text{(kg·cm)} \qquad (130) $$

(2) ${}_sM_{U1}$ shall be as given according to Table B3 in Appendix.

(3) ${}_sM_{U2}$ shall be as given by Eq. (131).

$$ {}_sM_{U2}={}_sM_{U3}-\frac{{}_sQ_U\cdot{}_bh}{2}+\frac{b_e\cdot F_B}{4}\left\{{}_bh^2-\left(\frac{{}_sQ_U}{b_e\cdot F_B}\right)^2\right\} \qquad \text{(kg·cm)} \qquad (131) $$

where F_B shall be as given by Eq. (132).

$$ F_B=\min\left(\sqrt{\frac{b}{b_e}}F_c,\ 12F_c,\ \frac{{}_wa\cdot{}_w\sigma_Y}{b_e\cdot x}\right) \qquad (\text{kg/cm}^2) \qquad (132) $$

where ${}_sM_{U3}$ shall be taken as ultimate flexural strength of the cross section underneath the base plate, as given according to Item 1 (2) of this article.

3. Ultimate shear strength of a bare type of column base may be taken as the sum of ultimate shear strength underneath the base plate and of the reinforced concrete portion surrounding the column base.

Article 37 Ultimate Strength of Structural Walls

1. Ultimate flexural strength of a structural wall including boundary columns shall be as given according to Table C1 in Appendix, where N is the axial force

acting at the wall center and ${}_{cs}F_c$ shall be taken equal to $0.75F_c$.

2. (1) Ultimate shear strength of a structural wall shall be as given by Eqs. (133) through (135).

$$ {}_wQ_U=\max({}_wQ_{U1},\ {}_wQ_{U2}) \qquad \text{(kg)} \qquad (133) $$

$$ {}_wQ_{U1}=r\cdot{}_wt\cdot l(1+\beta){}_wF_s \qquad \text{(kg)} \qquad (134) $$

$$ {}_wQ_{U2}=\max({}_wQ'_{U1},\ {}_wQ'_{U2})+{}_sQ_U \qquad \text{(kg)} \qquad (135) $$

where ${}_wF_s$ shall be as given by Eq. (136).

$$ {}_wF_s=\min\left(0.067F_c,\ 10+\frac{F_c}{50}\right) \qquad (\text{kg/cm}^2) \qquad (136) $$

(2) ${}_wQ'_{U1}$ and ${}_wQ'_{U2}$ shall be as given by Eqs. (I37) and (139), respectively.

$$ {}_wQ'_{U1}=r\cdot\min\{{}_wF'_s,\ ({}_wp\cdot{}_w\sigma_Y+{}_w\tau_U)\}{}_wt\cdot l'+\frac{1}{2}\min\left(\Sigma_{CS}Q_U,\ \Sigma_{BS}Q_U\frac{l}{h}\right) \qquad \text{(kg)} \qquad (137) $$

where ${}_wF'_c$ shall be as given by Eq. (138).

$$ {}_wF'_s=\min(0.25F_c,\ 3.6\sqrt{F_c}) \qquad (\text{kg/cm}^2) \qquad (138) $$

However, ${}_{cs}Q_U$ and ${}_{BS}Q_U$ shall, in general, be taken as 0, where the wall under consideration is not neighbored by another wall at either side.

$$ {}_wQ'_{U2}=\min\left(\Sigma_{CT}R_U,\ \Sigma_{BT}R_U\frac{l}{h}\right) \qquad \text{(kg)} \qquad (139) $$

However, ${}_{CT}R_U$ and ${}_{BT}R_U$ shall, in general, be taken as 0, where the wall under consideration is neighbored by another wall at either side.

(3) β and ${}_sQ_U$ shall be as given by Eqs. (98) and (140), respectively.

$$ {}_sQ_U=\begin{cases} {}_DA\cdot{}_s\sigma_Y\cdot\cos_D\theta : & \text{containing steel braces of flat bars or shapes} \quad \text{(kg)} \\ {}_swt\cdot l'\dfrac{{}_s\sigma_Y}{\sqrt{3}} : & \text{containing steel plates} \quad \text{(kg)} \end{cases} \qquad (140) $$

(4) ${}_w\tau_U$ shall be as given by Eq. (141).

$$ {}_w\tau_U=\min\left(\frac{{}_{BS}\beta_b}{{}_wt\cdot l^2}{}_{BS}M_U,\ \frac{{}_{CS}\beta_b}{{}_wt\cdot h^2}{}_{CS}M_U,\ \frac{{}_{BS}\beta_s}{{}_wt\cdot l}{}_{BS}Q_U,\ \frac{{}_{CS}\beta_s}{{}_wt\cdot h}{}_{CS}Q_U\right) \qquad (\text{kg/cm}^2) \qquad (141) $$

where ${}_{BS}\beta_b$, ${}_{BS}\beta_s$, ${}_{CS}\beta_b$ and ${}_{CS}\beta_s$ are constants related to the stress condition around the wall. ${}_{BS}M_U$, ${}_{CS}M_U$, ${}_{BS}Q_U$ and ${}_{CS}Q_U$ are ultimate flexural and shear strengths of beams and columns surrounding the wall, respectively, and shall be taken as the smaller of those of the upper and lower beams, and those of the columns on the right and left hand sides, respectively. When the wall under consideration is neighbored by other walls at both sides, $\frac{{}_{BS}\beta_b}{{}_wt\cdot l^2}{}_{BS}M_U$ and $\frac{{}_{BS}\beta_s}{{}_wt\cdot l}{}_{BS}Q_U$ for beams, and $\frac{{}_{cs}\beta_b}{{}_{w}t \cdot h^2}\, {}_{cs}M_U$ and $\frac{{}_{cs}\beta_b}{{}_{w}t \cdot h}\, {}_{cs}Q_U$ for columns need not be considered.

(5) ${}_{CT}R_U$ snd ${}_{BT}R_U$ shall be as given by Eqs. (142) and (143), respectively.

$$ {}_{CT}R_U = \min\left(\frac{2\,{}_{CT}M_U}{h_h},\ {}_{CT}Q_U\right) \quad (\mathrm{kg}) \qquad (142) $$

$$ {}_{BT}R_U = \min\left(\frac{2\,{}_{BT}M_U}{l_h},\ {}_{BT}Q_U\right) \quad (\mathrm{kg}) \qquad (143) $$

(6) Ultimate strengths of beams and columns adjacent to a structural wall shall be as given by considering tensile forces specified by Eq. (145) for beams and Eq. (146) for columns. However, h' shall be replaced by $h'/2$ for beams at the roof level, and l' by $l'/2$ for outer columns.

$$ {}_{BS}T_U = r \cdot \min\{({}_{w}F_s' - {}_{w}p \cdot {}_{w}\sigma_Y),\ {}_{w}\tau_U\}\,{}_{w}t \cdot h' \quad (\mathrm{kg}) \qquad (144) $$

$$ {}_{CS}T_U = r \cdot \min\{({}_{w}F_s' - {}_{w}p \cdot {}_{w}\sigma_Y),\ {}_{w}\tau_U\}\,{}_{w}t \cdot l' \quad (\mathrm{kg}) \qquad (145) $$

Section 4 Calculation of Lateral Load Resisting Capacity

Article 38 General

1. The lateral load resisting capacity for each story of a frame shall be as given by assuming that the collapse mechanism forms due to flexural and shear yielding of members, shear yielding of connections, shear and flexural yielding and rotation of structural walls.

2. General assumptions for the calculation of the lateral load resisting capacity shall conform to Article 12, Item 3.

3. Ultimate flexural strength of a column shall be as given for the axial force working at the formation of the collapse mechanism.

4. Location of plastic hinges in a beam shall be determined with intermediate loads taken into consideration.

5. Plastic hinges at beam and column ends shall form at the faces of beam- to-column connections.

6. For members in which shear failure occurs prior to flexural yielding, bending moments at member ends at the ultimate shear strength may be taken as the ultimate flexural strength.

7. The lateral load resisting capacity of a frame containing multi-story structural walls may be given as the sum of individual strengths of an unbraced frame and structural walls separated at intermediate points of beams connected to walls.

Chapter 6 Design of Composite Steel Tube and Concrete Structures

Section 1 General

Article 39 Scope

Requirements in this chapter apply to the allowable stress design and the investigation of the lateral load resisting capacity of composite steel tube and concrete structures. However, requirements in Chapters 4 and 5 apply to what is not specified in this chapter.

Section 2 Allowable Stress Design

Article 40 Calculation of Axial and Flexural Strengths of Columns

1. Calculation for a column with symmetrical cross section subjected to combined axial force and bending moment shall be as given according to this item.

(1) Calculation for a concrete-encased steel tubular column shall be as given by Eqs. (10) through (15).

(2) Calculation for a concrete-encased-and-filled steel tubular column shall be as given by Eqs. (146) through (150).

(i) When ${}_{r}N_t \leqq N \leqq {}_{r}N_b$

$$ \left.\begin{array}{ll} N = {}_{r}N & (\mathrm{kg}) \\ M \leqq {}_{s}M_0 + {}_{r}M & (\mathrm{kg\cdot cm}) \end{array}\right\} \qquad (146) $$

(ii) When ${}_{r}N_b < N \leqq {}_{r}N_b + {}_{c}N_c$

$$\left.\begin{aligned} N &= {}_rN_b + {}_cN && \text{(kg)} \\ M &\leq {}_sM_0 + {}_rM_b && \text{(kg·cm)} \end{aligned}\right\} \qquad (147)$$

(iii) When ${}_rN_b + {}_cN_c < N \leq {}_rN_c + {}_cN_c$

$$\left.\begin{aligned} N &= {}_cN_c + {}_rN && \text{(kg)} \\ M &\leq {}_sM_0 + {}_rM && \text{(kg·cm)} \end{aligned}\right\} \qquad (148)$$

(iv) When ${}_rN_c + {}_cN_c < N \leq {}_rN_c + {}_cN_c + {}_sN_c$

$$\left.\begin{aligned} N &= {}_rN_c + {}_cN_c + {}_sN && \text{(kg)} \\ M &\leq {}_sM && \text{(kg·cm)} \end{aligned}\right\} \qquad (149)$$

(v) When $N < {}_rN_t$ or $M < {}_sM_0$ with the axial force in tension

$$\left.\begin{aligned} N &\geq {}_rN_t + {}_sN && \text{(kg)} \\ M &= {}_sM && \text{(kg·cm)} \end{aligned}\right\} \qquad (150)$$

(3) Calculation for a concrete-filled steel tubular column shall be as given by Eqs. (151) through (153).

(i) When $0 \leq N \leq {}_cN_c$ or $M \geq {}_sM_0$

$$\left.\begin{aligned} N &= {}_cN && \text{(kg)} \\ M &\leq {}_sM_0 + {}_cM && \text{(kg·cm)} \end{aligned}\right\} \qquad (151)$$

(ii) When $N > N_c$ or $M < {}_sM_0$

$$\left.\begin{aligned} N &\leq {}_cN_c + {}_sN && \text{(kg)} \\ M &= {}_sM && \text{(kg·cm)} \end{aligned}\right\} \qquad (152)$$

(iii) When $N < 0$

$$\left.\begin{aligned} N &\geq {}_sN && \text{(kg)} \\ M &= {}_sM && \text{(kg·cm)} \end{aligned}\right\} \qquad (153)$$

(4) ${}_cN_c$ shall be as given by Eq. (154).

$${}_cN_c = {}_cA \cdot f_c' \qquad \text{(kg)} \qquad (154)$$

(5) ${}_rN_b$ and ${}_rM_b$ shall be as given by Eqs. (155) and (156), respectively.

$${}_rN_b = \frac{\dfrac{{}_mf_t}{n} + f_c'}{D - {}_rd_t} S_n \qquad \text{(kg)} \qquad (155)$$

$${}_rM_b = \left(\frac{I_n}{S_n} + \frac{D}{2} - \frac{D - {}_rd_t}{\dfrac{{}_mf_t}{n \cdot f_c'} + 1} \right) {}_rN_b \qquad \text{(kg·cm)} \qquad (156)$$

where S_n and I_n shall be as given by Eqs. (157) and (158), respectively.

$$S_n = \left\{ \frac{x_{n1b}^2}{2} + n \cdot {}_mp_c (x_{n1b} - {}_rd_{c1}) - n \cdot {}_mp_t (1 - {}_rd_{t1} - x_{n1b}) \right\} bD^2 \qquad (\text{cm}^3) \qquad (157)$$

$$I_n = \left\{ \frac{x_{n1b}^3}{3} + n \cdot {}_mp_c (x_{n1b} - {}_rd_{c1})^2 + n \cdot {}_mp_t (1 - {}_rd_{t1} - x_{n1b})^2 \right\} bD^3 \qquad (\text{cm}^4) \qquad (158)$$

(6) x_{n1b} shall be as given by Eq. (159).

$$x_{n1b} = \frac{1 - {}_rd_{t1}}{1 + \dfrac{{}_mf_t}{n \cdot f_c'}} \qquad (159)$$

(7) ${}_rN$, ${}_rM$, ${}_cN$, and ${}_cM$ shall be as given according to Standards for Structural Calculation of Reinforced Concrete Structures. However, ${}_rN$ and ${}_rM$ of a hollow symmetrical section shall be as given according to Tables Dl and D2 in Appendix, and ${}_cN$ and ${}_cM$ of the filled concrete portion according to Table D3 in Appendix.

(8) f_c' shall be as given by Eq. (160).

$$f_c' = \begin{cases} f_c(1 - 4\,{}_sp) & : \text{covering reinforced concrete portion} \\ f_c & : \text{filled concrete portion} \end{cases} \qquad (\text{kg/cm}^2) \qquad (160)$$

2. Calculation for a column subjected to combined axial force and bending moment may be as given by Eq. (161), instead of Eqs. (10) through (15), or Eqs. (146) through (153).

$$\left.\begin{aligned} N &= {}_sN + {}_rN + {}_cN && \text{(kg)} \\ M &\leq {}_sM + {}_rM + {}_cM && \text{(kg·cm)} \end{aligned}\right\} \qquad (161)$$

3. Calculation for a column with unsymmetrical cross section may be as given by Eqs. (10) through (15), or Eqs. (146) though (153) by replacing the section by conservatively modified symmetrical cross section, or by Eq.(161) taking the unsymmetry into consideration.

4. Calculation for a column subjected to combined axial force and biaxial bending moments may be as given by Eq. (162).

$$\left.\begin{aligned} N &= {}_sN + {}_rN + {}_cN && \text{(kg)} \\ M_x &\leq {}_sM_x + {}_rM_x + {}_cM_x && \text{(kg·cm)} \\ M_y &\leq {}_sM_y + {}_rM_y + {}_cM_y && \text{(kg·cm)} \end{aligned}\right\} \qquad (162)$$

5. Calculation for a column of which effective buckling length exceeds 12 times the depth of the cross section shall conform to the requirements in this item. However, a concrete-encased-and-filled steel tubular column shall be treated as a concrete-encased steel tubular column, ignoring filled concrete inside the steel tube.

(1) Calculation for a concrete-encased steel tubular column shall be as given by Eqs. (32) through (35).

(2) Calculation for a concrete-filled steel tubular column shall be as given by Eqs. (163) and (164).

(i) When $N \leqq {}_cN_c$ or $M \geqq {}_sM_0\left(1-\frac{{}_c\nu \cdot {}_cN_c}{N_k}\right)$

$$\left.\begin{aligned} &N={}_cN && \text{(kg)} \\ &M \leqq {}_sM_0\left(1-\frac{{}_c\nu \cdot {}_cN}{N_k}\right)+{}_cM && \text{(kg·cm)} \end{aligned}\right\} \quad (163)$$

(ii) When $N > {}_cN_c$ or $M < {}_sM_0\left(1-\frac{{}_c\nu \cdot {}_cN_c}{N_k}\right)$

$$\left.\begin{aligned} &N \leqq {}_cN_c+{}_sN && \text{(kg)} \\ &M={}_sM\left(1-\frac{{}_c\nu \cdot {}_cN_c}{N_k}\right) && \text{(kg·cm)} \end{aligned}\right\} \quad (164)$$

where ${}_c\nu$ shall be taken as 3.0 and 1.5 for the long-term and the short-term loading conditions, respectively.

(3) ${}_sN$ and ${}_sN_c$ are allowable compressive strengths of the steel portion as a long column, and shall be as given according to Design Standards for Steel Structures taking the slenderness effect into consideration.

(4) ${}_cN$ and ${}_cN_c$ are allowable compressive strengths of the filled concrete portion as a long column, and shall be as given according to Item 1 (7) of this article for the cross section subjected to bending moment equal to ${}_cM$ multiplied by ${}_c\delta$ specified by Eq. (165) and axial compression ${}_cN$. However, end eccentricities not less than 5% of the concrete depth shall be considered in the calculation described above.

$${}_c\delta=\frac{1}{1-\dfrac{{}_c\nu \cdot {}_cN}{{}_cN_k}} \quad (165)$$

where ${}_cN_k$ shall be as given by Eq. (166).

$${}_cN_k=\frac{\pi^2 {}_cE \cdot {}_cI}{5 l_k^2} \quad \text{(kg)} \quad (166)$$

6. Compression force acting on a column under the earthquake loading shall be limited according to Article 17, Item 6.

Article 41 Calculation of Shear Strength of Members

1. Calculation of shear force of a composite steel tube and concrete column shall be as given by Eqs. (54) through (56). However, calculation for a concrete-filled steel tubular column shall be as given by Eq. (55).

2.(1) Allowable shear strength for the long-term loading condition shall be as given by Eq. (57), where ${}_rQ_{AL}$ shall be as given by Eq. (167).

$${}_rQ_{AL}=b' \cdot {}_rj \cdot \alpha \cdot f_s \quad \text{(kg)} \quad (167)$$

where α, β and ${}_rj$ shall be as given by Eqs. (53), (168), and (169), respectively.

$$\beta=\frac{7.5 {}_sA}{b' \cdot {}_rj} \quad (168)$$

$${}_rj=0.75D \quad \text{(cm)} \quad (169)$$

(2) Allowable shear strength of a steel tube for the short-term loading condition ${}_sQ_A$ shall be as given by Eq. (170).

$${}_sQ_A=\frac{{}_sA \cdot {}_sf_s}{2} \quad \text{(kg)} \quad (170)$$

(3) Allowable shear strength of the covering reinforced concrete portion for the short-term loading condition ${}_rQ_{AS}$ shall be as given by Eq. (171).

$${}_rQ_{AS}=b' \cdot {}_rj(f_s+0.5 {}_wp \cdot {}_wf_t) \quad \text{(kg)} \quad (171)$$

where ${}_wp$ shall be taken equal to 1.2%, when it exceeds 1.2%.

3. Design shear force for the steel tube and the covering reinforced concrete portion for the short-term loading condition, ${}_sQ_d$ and ${}_rQ_d$, shall be as given according to Article 18, Item 3 (5). However, ${}_rM_1$ and ${}_rM_2$ shall be as given for a column subjected to the compression force specified in Table 19.

Table 19 Compression Force for the Calculation of Ultimate Flexural Strength (kg)

	Range of compression force	Compression force
Concrete-encased type	$N \leqq {}_rN_c$	N
	$N > {}_rN_c$	${}_rN_c$
Concrete-encased-and-filled type	$N \leqq {}_rN_b$	N
	${}_rN_b < N \leqq {}_rN_b+{}_cN_c$	$N-{}_cN$
	${}_rN_b+{}_cN_c < N \leqq {}_rN_c+{}_cN_c$	$N-{}_cN_c$
	$N > {}_rN_c+{}_cN_c$	${}_rN_c$

Article 42 Bond between Steel Tube and Concrete

Bond between steel tube and concrete shall be investigated, where a part of the shear force in a beam is transferred as the compression force in the filled concrete portion of a column.

Article 43 Beam-to-Column Connections

1. Calculation of shear force of a connection panel surrounded by columns and beams shall be as given according to this article.

2. (1) Calculation for concrete-encased-and-filled and concrete-encased steel tubular connections for the long-term loading condition shall be as given by Eq. (75), and for the short-term loading condition by Eq. (76), where $_cV$, $_cV_e$ and β shall be as given according to Table 20, and $_j\delta$ according to Table 16.

(2) Calculation for a concrete-filled steel tubular connection shall be as given by Eq. (172).

$$2f_s\cdot{}_j\beta\cdot{}_cV+{}_sV\cdot{}_sf_s\geqq\frac{h'}{h}({}_BM_1+{}_BM_2) \qquad (\text{kg}\cdot\text{cm}) \qquad (172)$$

where $_cV$ and $_j\beta$ shall be as given according to Tables 20 and 21, respectively.

Table 20 Values of $_cV$, $_cV_e$, $_sV$ and β

Type of connection		$_cV$	$_cV_e$	$_sV$	β
Concrete-encased-and-filled	With SRC or RC beams	$_cb\cdot{}_{mB}d\cdot{}_{mc}d$	$\frac{_Bb+{}_cb}{2}{}_{mB}d\cdot{}_{mc}d$	$\frac{_sA}{2}\cdot{}_{sB}d$	$7.5\frac{_sA}{_cb\cdot{}_{mc}d}$
	With S beams	$_cb\cdot{}_{sB}d\cdot{}_{mc}d$	$\frac{_cb}{2}{}_{sB}d\cdot{}_{mc}d$		
Concrete-encased	With SRC or RC beams	$(_cb-{}_sD)_{mB}d\cdot{}_{mc}d$	$\frac{_Bb+(_cb-{}_sD)}{2}{}_{mB}d\cdot{}_{mc}d$		$7.5\frac{_sA}{(_cb-{}_sD)_{mc}d}$
	With S beams	$(_cb-{}_sD)_{sB}d\cdot{}_{mc}d$	$\frac{_cb-{}_sD}{2}{}_{sB}d\cdot{}_{mc}d$		
Concrete-filled	With S beams	$_cA\cdot{}_{sB}d$	—		—

Table 21 Values of $_j\beta$

Shape of steel tube	$_j\beta$
Circular	$2\frac{_sD}{_{sB}d}$ but not larger than 4
Rectangular	$2.5\frac{_sD}{_{sB}d}$ but not larger than 4

3. A stiffener ring of a connection may be proportioned considering the effect of the filled concrete and the steel tube wall confining the deformation of each other.

Article 44 Column Bases

b_e of an embedded type of a column base shall be determined according to Table 22.

Table 22 Effective Width of Embedded Type of Column Base (cm)

Cross section of steel tube	b_e
Rectangular	$3.6{}_st$
Circular	$0.4{}_sD$

Section 3 Calculation of Lateral Load Resisting Capacity

Article 45 Ultimate Flexural Strength of Members

Ultimate flexural strength of a composite concrete and steel tubular member shall be as given by Eqs. (108) through (110), or by Eqs. (115) and (116).

(1) $_cM_U$ shall be as given according to Table El in Appendix, where $_c\gamma_U$ shall be as given by Eq. (173).

$$_c\gamma_U=\begin{cases}0.85-0.6{}_sp & \text{: covering reinforced concrete portion}\\ 0.85 & \text{: filled concrete portion}\end{cases} \qquad (173)$$

(2) $_sM_U$ shall be as given according to Table B3 in Appendix.

Article 46 Ultimate Shear Strength of Members

1. Ultimate shear strength of a composite concrete and steel tubular member shall be as given by Eq. (119), or by Eqs. (174) and (175).

(i) Calculation for a concrete-encased steel tubular column shall be as given by Eq. (119).

(ii) Calculation for a concrete-filled steel tubular column shall be as given by Eq. (174).

$$Q_U={}_cQ_U+{}_sQ_U \qquad (\text{kg}) \qquad (174)$$

(iii) Calculation for a concrete-encased-and-filled steel tubular column shall be as given by Eq. (175).

$$Q_U={}_rQ_U+{}_cQ_U+{}_sQ_U \qquad (\text{kg}) \qquad (175)$$

2. $_rQ_U$ shall be as given by Eq. (120). However, $_rQ_{sU}$ shall be as given by Eq.

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(176).

$$ {}_{r}Q_{sU} = b' \cdot {}_{r}j(0.5F_s \cdot {}_{r}\alpha + 0.5{}_{w}p \cdot {}_{w}\sigma_Y) \qquad \text{(kg)} \qquad (176) $$

where F_s and ${}_{r}\alpha$ shall be as given by Eqs. (124) and (45), respectively.

3. ${}_{c}Q_U$ shall be as given by Eq. (177).

$$ {}_{c}Q_U = \Sigma \frac{{}_{c}M_U}{l'} \qquad \text{(kg)} \qquad (177) $$

4. ${}_{r}M_U$ may be as given according to Tables E1 and B2 in Appendix, and ${}_{c}M_U$ according to Table Bl. However, the compression force shall be as given in Table 23.

Table 23 Compression Force for the Calculation of Ultimate Flexural Strength (kg)

	Range of compression force	Compression force
Concrete-encased type	$N \leqq {}_{c}N$	N
	$N > {}_{c}N$	${}_{c}N_c$
Concrete-encased-and-filled type	$N \leqq {}_{r}N_b$	N
	${}_{r}N_b < N \leqq {}_{r}N_b + {}_{c}N_c$	$N - {}_{c}N$
	${}_{r}N_b + {}_{c}N_c < N \leqq {}_{r}N_c + {}_{c}N_c$	$N - {}_{c}N_c$
	$N > {}_{r}N_c + {}_{c}N_c$	${}_{r}N_c$

5. ${}_{s}Q_U$ shall be as given by Eq. (125). However, ${}_{s}Q_{sU}$ shall be as given by Eq. (178).

$$ {}_{s}Q_{sU} = \frac{{}_{s}A}{2} \cdot \frac{{}_{s}\sigma_Y}{\sqrt{3}} \qquad \text{(kg)} \qquad (178) $$

Article 47 Ultimate Shear Strength of Beam-to-Column Connections

(1) Calculation for concrete-encased-and-filled and concrete-encased steel tubular connections shall be as given by Eq. (128). However, ${}_{c}V_e$ and ${}_{s}V$ shall be determined according to Table 20, and ${}_{J}\delta$ according to Table 16.

(2) Calculation for a concrete-filled steel tubular connection shall be as given by Eq. (179).

$$ {}_{J}M_U = {}_{c}V \cdot {}_{J}F_s \cdot {}_{J}\beta + 1.2{}_{s}V \frac{{}_{s}\sigma_Y}{\sqrt{3}} \qquad \text{(kg·cm)} \qquad (179) $$

where ${}_{s}V$ and ${}_{c}V$ shall be determined according to Table 20, and ${}_{J}\beta$ according to Table 21. ${}_{J}F_s$ shall be as given by Eq. (129).

Appendix A Allowable Strength of Column Bases

Table A1 ${}_{b}N - {}_{b}M$ Relation

Range of ${}_{b}N$	${}_{b}M$
$0 \leqq {}_{b}N \leqq \frac{{}_{b}N_c}{2}$	$\frac{{}_{b}N}{{}_{b}N_c}\left(\frac{3}{4} - \frac{{}_{b}N}{{}_{b}N_c}\right)\frac{2{}_{b}b \cdot {}_{b}D^2 \cdot f_c}{3}$
$\frac{{}_{b}N_c}{2} < {}_{b}N \leqq {}_{b}N_c$	$\left(1 - \frac{{}_{b}N}{{}_{b}N_c}\right)\frac{{}_{b}b \cdot {}_{b}D^2 \cdot f_c}{6}$

Table A2 ${}_{a}N - {}_{a}M$ Relation

Range of ${}_{a}N$	${}_{a}M$
${}_{a}N_t \leqq {}_{a}N < \frac{{}_{a}N_t}{2}$	$2\left(1 - \frac{{}_{a}N}{{}_{a}N_t}\right){}_{a}M_o$
$\frac{{}_{a}N_t}{2} < {}_{a}N \leqq 0$	$2\frac{{}_{a}N}{{}_{a}N_t}{}_{a}M_o$

Table A3 Values of ${}_{b}N_c$, ${}_{a}N_t$ and ${}_{a}M_o$

${}_{b}N_c$	${}_{b}b \cdot {}_{b}D \cdot f_c$
${}_{a}N_t$	$-{}_{a}A \cdot {}_{a}f_t$
${}_{a}M_o$	${}_{a}A \cdot {}_{a}d \cdot {}_{a}f_t / 4$

Table A4 ${}_{c}N$-${}_{c}M$ Relation

Range of ${}_{c}N$	x_n	${}_{c}N/(bD \cdot f_c')$	${}_{c}M/(bD^2 \cdot f_c')$
$0 \leqq {}_{c}N < \frac{{}_{c}N_c}{2}$	$0 \leqq x_{n1} < \frac{D - {}_{b}D}{2D}$	$\frac{x_{n1}}{2}$	$\frac{x_{n1}}{12}(3 - 2x_{n1})$
	$\frac{D - {}_{b}D}{2D} \leqq x_{n1} < \frac{D + {}_{b}D}{2D}$	$\frac{1}{2x_{n1}}\left\{x_{n1}^2 - \left(\frac{{}_{b}b}{b}\right) \times \left(x_{n1} - \frac{D - {}_{b}D}{2D}\right)^2\right\}$	$\frac{1}{12x_{n1}}\left\{x_{n1}^2(3 - 2x_{n1}) - \left(\frac{{}_{b}b}{b}\right) \times \left(x_{n1} - \frac{D - {}_{b}D}{2D}\right)^2\left(1 - 2x_{n1} + \frac{{}_{b}D}{D}\right)\right\}$
	$\frac{D + {}_{b}D}{2D} \leqq x_{n1} \leqq 1$	$\frac{1}{2x_{n1}}\left\{x_{n1}^2 - \left(\frac{{}_{b}b}{b}\right) \times \frac{{}_{b}D}{D}(2x_{n1} - 1)\right\}$	$\frac{1}{12x_{n1}}\left\{x_{n1}^2(3 - 2x_{n1}) - \left(\frac{{}_{b}b}{b}\right)\left(\frac{{}_{b}D}{D}\right)^3\right\}$
$\frac{{}_{c}N_c}{2} \leqq {}_{c}N \leqq {}_{c}N_c$	${}_{c}M$		
	$\left(1 - \frac{{}_{c}N}{{}_{c}N_c}\right)\frac{bD^3 - {}_{b}b \cdot {}_{b}D^3}{6D}f_c'$		

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Table A5 ${}_{m}N$-${}_{m}M$ Relation

Range of ${}_{m}N$	${}_{m}M$
$0 \leqq {}_{m}N \leqq {}_{m}N_c$	$\left(1-\frac{{}_{m}N}{{}_{m}N_c}\right){}_{m}M_o$
${}_{m}N_t \leqq {}_{m}N < 0$	$\left(1-\frac{{}_{m}N}{{}_{m}N_t}\right){}_{m}M_o$

Table A6 Values of ${}_{c}N_c$, ${}_{m}N_c$, ${}_{m}N_t$ and ${}_{m}M_o$

${}_{c}N_c$	(b D $- {}_{b}b \cdot {}_{b}D) f'_c$
${}_{m}N_c$	${}_{m}A \cdot {}_{m}f_c$
${}_{m}N_t$	$- {}_{m}A \cdot {}_{m}f_t$
${}_{m}M_o$	${}_{m}a_t \cdot {}_{m}d \cdot {}_{m}f_t$

Appendix B Ultimate Flexural Strength of Members

Table B1 Ultimate Flexural Strength of Concrete Portion

Rectangular section	${}_{c}M_U/(bD^2 \cdot F_c)$	
	$\frac{1}{2}\frac{{}_{c}N_U}{bD \cdot F_c}\left(1-\frac{1}{{}_{c}\gamma_U}\frac{{}_{c}N_U}{bD \cdot F_c}\right)$	
Ciruclar section	${}_{c}N_U/({}_{c}D^2 \cdot F_c)$	${}_{c}M_U/({}_{c}D^3 \cdot F_c)$
	$\frac{{}_{c}\gamma_U}{4}(\theta_n - \sin\theta_n \cdot \cos\theta_n)$	$\frac{{}_{c}\gamma_U}{12}\sin^3\theta_n$

Table B2 Ultimate Flexural Strength of Longitudinal Reinforcement

	Range of axial force	${}_{m}M_U$
Symmetrical bar arrangement	$0 \leqq {}_{m}N_U \leqq 2{}_{m}a_t \cdot {}_{m}\sigma_Y$	${}_{m}d({}_{m}a_t \cdot {}_{m}\sigma_Y - {}_{m}N_U/2)$
	$-2{}_{m}a_t \cdot {}_{m}\sigma_Y \leqq {}_{m}N_U < 0$	${}_{m}d({}_{m}a_t \cdot {}_{m}\sigma_Y + {}_{m}N_U/2)$
Symmetrical bar arrangerent with intermediate bars	${}_{m}a_m \cdot {}_{m}\sigma_Y \leqq {}_{m}N_U \leqq (2{}_{m}a_t + {}_{m}a_m){}_{m}\sigma_Y$	${}_{m}d\left\{{}_{m}a_t \cdot {}_{m}\sigma_Y - \frac{1}{2}({}_{m}N_U - {}_{m}a_m \cdot {}_{m}\sigma_Y)\right\}$
	$-{}_{m}a_m \cdot {}_{m}\sigma_Y \leqq {}_{m}N_U \leqq {}_{m}\sigma_m \cdot {}_{m}\sigma_Y$	${}_{m}d \cdot {}_{m}a_t \cdot {}_{m}\sigma_Y$
	$-(2{}_{m}a_t + {}_{m}a_m){}_{m}\sigma_Y \leqq {}_{m}N_U \leqq -{}_{m}a_m \cdot {}_{m}\sigma_Y$	${}_{m}d\left\{{}_{m}a_t \cdot {}_{m}\sigma_Y + \frac{1}{2}({}_{m}N_U + {}_{m}a_m \cdot {}_{m}\sigma_Y)\right\}$

Table B3 Ultimate Flexural Strength of Steel Portion

	Range of axial force	${}_{s}M_U$
Open-web section under strong-axis bending	$0 \leqq {}_{s}N_U \leqq {}_{s}A \cdot {}_{s}\sigma_Y$	${}_{s}Z_P \cdot {}_{s}\sigma_Y - {}_{s}N_U \cdot {}_{s}d/2$
	$-{}_{s}A \cdot {}_{s}\sigma_Y \leqq {}_{s}N_U < 0$	${}_{s}Z_P \cdot {}_{s}\sigma_Y + {}_{s}N_U \cdot {}_{s}d/2$
Cross-shaped open-web section	${}_{s}a_m \cdot {}_{s}\sigma_Y < {}_{s}N_U \leqq {}_{s}A \cdot {}_{s}\sigma_Y$	${}_{s}Z_P \cdot {}_{s}\sigma_Y - \frac{{}_{s}d}{2}({}_{s}N_U - {}_{s}a_m \cdot {}_{s}\sigma_Y)$
	$-{}_{s}a_m \cdot {}_{s}\sigma_Y \leqq {}_{s}N_U \leqq {}_{s}a_m \cdot {}_{s}\sigma_Y$	${}_{s}Z_P \cdot {}_{s}\sigma_Y$
	$-{}_{s}A \cdot {}_{s}\sigma_Y \leqq {}_{s}N_U < -{}_{s}a_m \cdot {}_{s}\sigma_Y$	${}_{s}Z_P \cdot {}_{s}\sigma_Y + \frac{{}_{s}d}{2}({}_{s}N_U + {}_{s}a_m \cdot {}_{s}\sigma_Y)$
Full-web section under strong-axis bending and rectangular tube	$\frac{{}_{s}a_w}{2} \cdot {}_{s}\sigma_Y < {}_{s}N_U \leqq {}_{s}A \cdot {}_{s}\sigma_Y$	${}_{s}Z_P \cdot {}_{s}\sigma_Y - \frac{{}_{s}d}{2}\left({}_{s}N_U - \frac{1}{2}{}_{s}a_w \cdot {}_{s}\sigma_Y\right)$
	$-\frac{{}_{s}a_w}{2} \cdot {}_{s}\sigma_Y \leqq {}_{s}N_U \leqq \frac{{}_{s}a_w}{2} \cdot {}_{s}\sigma_Y$	${}_{s}Z_P \cdot {}_{s}\sigma_Y$
	$-{}_{s}A \cdot {}_{s}\sigma_Y \leqq {}_{s}N_U < -\frac{{}_{s}a_w}{2} \cdot {}_{s}\sigma_Y$	${}_{s}Z_P \cdot {}_{s}\sigma_Y + \frac{{}_{s}d}{2}\left({}_{s}N_U + \frac{1}{2}{}_{s}a_w \cdot {}_{s}\sigma_Y\right)$
Full-web section under weak-axis bending	$({}_{s}a_f + {}_{s}a_w){}_{s}\sigma_Y < {}_{s}N_U \leqq {}_{s}A \cdot {}_{s}\sigma_Y$	$2{}_{s}Z_P \cdot {}_{s}\sigma_Y - \frac{b_f}{2}({}_{s}N_U - {}_{s}a_w \cdot {}_{s}\sigma_Y)$
	$-({}_{s}a_f + {}_{s}a_w){}_{s}\sigma_Y \leqq {}_{s}N_U \leqq ({}_{s}a_f + {}_{s}a_w){}_{s}\sigma_Y$	${}_{s}Z_P \cdot {}_{s}\sigma_Y$
	$-{}_{s}A \cdot {}_{s}\sigma_Y \leqq {}_{s}N_U < -({}_{s}a_f + {}_{s}a_w){}_{s}\sigma_Y$	$2{}_{s}Z_P \cdot {}_{s}\sigma_Y + \frac{b_f}{2}({}_{s}N_U + {}_{s}a_w \cdot {}_{s}\sigma_Y)$
Cross-shaped full-web section	$\left(2{}_{s}a_f + \frac{3{}_{s}a_w}{2}\right){}_{s}\sigma_Y < {}_{s}N_U \leqq {}_{s}A \cdot {}_{s}\sigma_Y$	${}_{s}d \cdot \frac{{}_{s}A}{2} \cdot {}_{s}\sigma_Y - \frac{{}_{s}d}{2} \cdot {}_{s}N_U$
	$\left({}_{s}a_f + \frac{3}{2}{}_{s}a_w\right){}_{s}\sigma_Y < {}_{s}N_U \leqq \left(2{}_{s}a_f + \frac{3{}_{s}a_w}{2}\right){}_{s}\sigma_Y$	${}_{s}Z_P \cdot {}_{s}\sigma_Y - \frac{b_f}{2}\left({}_{s}N_U - \frac{3}{2}{}_{s}a_w \cdot {}_{s}\sigma_Y - {}_{s}a_f \cdot {}_{s}\sigma_Y\right)$
	$-\left({}_{s}a_f + \frac{3}{2}{}_{s}a_w\right){}_{s}\sigma_Y \leqq {}_{s}N_U \leqq \left({}_{s}a_f + \frac{3}{2}{}_{s}a_w\right){}_{s}\sigma_Y$	${}_{s}Z_P \cdot {}_{s}\sigma_Y$
	$-\left(2{}_{s}a_f + \frac{3{}_{s}a_w}{2}\right){}_{s}\sigma_Y \leqq {}_{s}N_U < -\left({}_{s}a_f + \frac{3}{2}{}_{s}a_w\right){}_{s}\sigma_Y$	${}_{s}Z_P \cdot {}_{s}\sigma_Y + \frac{b_f}{2}\left({}_{s}N_U + \frac{3}{2}{}_{s}a_w \cdot {}_{s}\sigma_Y + {}_{s}a_f \cdot {}_{s}\sigma_Y\right)$
	$-{}_{s}A \cdot {}_{s}\sigma_Y \leqq {}_{s}N_U < -\left(2{}_{s}a_f + \frac{3{}_{s}a_w}{2}\right){}_{s}\sigma_Y$	${}_{s}d \cdot \frac{{}_{s}A}{2} \cdot {}_{s}\sigma_Y + \frac{{}_{s}d}{2} \cdot {}_{s}N_U$
Circular tube	$0.2{}_{s}A \cdot {}_{s}\sigma_Y < {}_{s}N_U \leqq {}_{s}A \cdot {}_{s}\sigma_Y$	$1.25\left(1 - \frac{{}_{s}N_U}{{}_{s}A \cdot {}_{s}\sigma_Y}\right){}_{s}Z_P \cdot {}_{s}\sigma_Y$
	$-0.2{}_{s}A \cdot {}_{s}\sigma_Y \leqq {}_{s}N_U \leqq 0.2{}_{s}A \cdot {}_{s}\sigma_Y$	${}_{s}Z_P \cdot {}_{s}\sigma_Y$
	$-{}_{s}A \cdot {}_{s}\sigma_Y \leqq {}_{s}N_U < -0.2{}_{s}A \cdot {}_{s}\sigma_Y$	$1.25\left(1 + \frac{{}_{s}N_U}{{}_{s}A \cdot {}_{s}\sigma_Y}\right){}_{s}Z_P \cdot {}_{s}\sigma_Y$

Table B4 Ultimate Flexural Strength of Reinforced Concrete Portion

Range of axial force	${}_{r}M_U$
${}_{c}\gamma_U\cdot F_c\cdot bD<{}_{r}N_U\leqq{}_{c}\gamma_U\cdot F_c\cdot bD+2{}_{m}a_t\cdot{}_{r}\sigma_Y$	${}_{m}a_t\cdot{}_{m}\sigma_Y\cdot{}_{m}d-\frac{{}_{m}d}{2}({}_{r}N_U-{}_{c}\gamma_U\cdot F_c\cdot bD)$
$0\leqq{}_{r}N_U\leqq{}_{c}\gamma_U\cdot F_c\cdot bD$	${}_{m}a_t\cdot{}_{m}\sigma_Y\cdot{}_{m}d+\frac{{}_{r}N_U\cdot D}{2}\left(1-\frac{{}_{r}N_U}{{}_{c}\gamma_U\cdot F_c\cdot bD}\right)$
$-2{}_{m}a_t\cdot{}_{m}\sigma_Y\leqq{}_{r}N_U<0$	${}_{m}a_t\cdot{}_{m}\sigma_Y\cdot{}_{m}d+\frac{{}_{m}d}{2}{}_{r}N_U$

Table B5 Ultimate Flexural Strength of Steel Reinforced Concrete Member with Full-web Type Steel Section

Range of axial force	M_U
$\frac{{}_{s}a_w}{2}{}_{s}\sigma_Y+{}_{c}\gamma_U\cdot F_c\cdot bD<N_U$ $\leqq{}_{c}\gamma_U\cdot F_c\cdot bD+{}_{s}A\cdot{}_{s}\sigma_Y+2{}_{m}a_t\cdot{}_{m}\sigma_Y$	$\left\{\frac{{}_{s}Z_P\cdot{}_{s}\sigma_Y+{}_{m}a_t\cdot{}_{m}\sigma_Y\cdot{}_{m}d}{\left(-{}_{s}A+\frac{{}_{s}a_w}{2}\right){}_{s}\sigma_Y-2{}_{m}a_t\cdot{}_{m}\sigma_Y}\right\}(N_U-{}_{s}A\cdot{}_{s}\sigma_Y-2{}_{m}a_t\cdot{}_{m}\sigma_Y-{}_{c}\gamma_U\cdot F_c\cdot bD)$
$\frac{{}_{s}a_w}{2}{}_{s}\sigma_Y+\frac{{}_{c}\gamma_U}{2}F_c\cdot bD<N_U$ $\leqq\frac{{}_{s}a_w}{2}{}_{s}\sigma_Y+{}_{c}\gamma_U\cdot F_c\cdot bD$	$\frac{D}{2}\left(N_U-\frac{{}_{s}a_w}{2}{}_{s}\sigma_Y\right)\left(1-\frac{N_U-\frac{{}_{s}a_w}{2}{}_{s}\sigma_Y}{{}_{c}\gamma_U\cdot F_c\cdot bD}\right)+{}_{s}Z_P\cdot{}_{s}\sigma_Y+{}_{m}a_t\cdot{}_{m}\sigma_Y\cdot{}_{m}d$
$-\frac{{}_{s}a_w}{2}{}_{s}\sigma_Y+\frac{{}_{c}\gamma_U}{2}F_c\cdot bD\leqq N_U$ $\leqq\frac{{}_{s}a_w}{2}{}_{s}\sigma_Y+\frac{{}_{c}\gamma_U}{2}F_c\cdot bD$	$\frac{{}_{c}\gamma_U}{8}bD^2\cdot F_c+{}_{s}Z_P\cdot{}_{s}\sigma_Y+{}_{m}a_t\cdot{}_{m}\sigma_Y\cdot{}_{m}d$
$-\frac{{}_{s}a_w}{2}{}_{s}\sigma_Y\leqq N_U$ $<-\frac{{}_{s}a_w}{2}{}_{s}\sigma_Y+\frac{{}_{c}\gamma_U}{2}F_c\cdot bD$	$\frac{D}{2}\left(N_U+\frac{{}_{s}a_w}{2}{}_{s}\sigma_Y\right)\left(1-\frac{N_U+\frac{{}_{s}a_w}{2}\cdot{}_{s}\sigma_Y}{{}_{c}\gamma_U\cdot F_c\cdot bD}\right)+{}_{s}Z_P\cdot{}_{s}\sigma_Y+{}_{m}a_t\cdot{}_{m}\sigma_Y{}_{m}d$
$-{}_{s}A\cdot{}_{s}\sigma_Y-2{}_{m}a_t\cdot{}_{m}\sigma_Y\leqq N_U$ $<-\frac{{}_{s}a_w}{2}{}_{s}\sigma_Y$	$\left\{\frac{{}_{s}Z_P\cdot{}_{s}\sigma_Y+{}_{m}a_t\cdot{}_{m}\sigma_Y\cdot{}_{m}d}{\left({}_{s}A-\frac{{}_{s}a_w}{2}\right){}_{s}\sigma_Y+2{}_{m}a_t\cdot{}_{m}\sigma_Y}\right\}(N_U+{}_{s}A\cdot{}_{s}\sigma_Y+2{}_{m}a_t\cdot{}_{m}\sigma_Y)$

Appendix C Ultimate Flexural Strength of Structural Walls

Table C1 Ultimate Flexural Strength of Structural Walls

Range of axial force	Ultimate flexural strength
$-2({}_{scs}A\cdot{}_{s}\sigma_Y+{}_{mcs}A\cdot{}_{m}\sigma_Y)-{}_{mw}A\cdot{}_{w}\sigma_Y\leqq N_U$ $<{}_{ccs}A\cdot{}_{cs}F_c-{}_{mw}A\cdot{}_{w}\sigma_Y$	$\left(\frac{N_U}{2}+{}_{scs}A\cdot{}_{s}\sigma_Y+{}_{mcs}A\cdot{}_{m}\sigma_Y+\frac{{}_{mw}A}{2}{}_{w}\sigma_Y\right){}_{w}l$
${}_{cs}A\cdot{}_{cs}F_c-{}_{mw}A\cdot{}_{w}\sigma_Y\leqq N_U<{}_{cs}F_c-{}_{mw}A\cdot{}_{w}\sigma_Y$ $+\frac{{}_{w}t\cdot{}_{w}l'\cdot F_c}{2}$	$(N_U-{}_{cs}A\cdot{}_{cs}F_c+{}_{mw}A\cdot{}_{w}\sigma_Y)$ $\times\left(1-\frac{N_U-{}_{cs}A\cdot{}_{cs}F_c+{}_{mw}A\cdot{}_{w}\sigma_Y}{{}_{w}t\cdot{}_{w}l'\cdot F_c}\right)\frac{{}_{w}l'}{2}$ $+\left({}_{scs}A\cdot{}_{s}\sigma_Y+{}_{mcs}A\cdot{}_{m}\sigma_Y+\frac{{}_{cs}A\cdot{}_{cs}F_c}{2}\right){}_{w}l$
${}_{cs}A\cdot{}_{cs}F_c-{}_{mw}A\cdot{}_{w}\sigma_Y+\frac{{}_{w}t\cdot{}_{w}l'\cdot F_c}{2}\leqq N_U$ $\leqq{}_{cs}A\cdot{}_{cs}F_c+{}_{mw}A\cdot{}_{w}\sigma_Y+\frac{{}_{w}t\cdot{}_{w}l'\cdot F_c}{2}$	$\frac{{}_{w}t\cdot{}_{w}l'^2\cdot F_c}{8}+\left({}_{scs}A\cdot{}_{s}\sigma_Y+{}_{mcs}A\cdot{}_{m}\sigma_Y+\frac{{}_{cs}A\cdot{}_{cs}F_c}{2}\right){}_{w}l$
${}_{cs}A\cdot{}_{cs}F_c+{}_{mw}A\cdot{}_{w}\sigma_Y+\frac{{}_{w}t\cdot{}_{w}l'\cdot F_c}{2}<N_U$ $\leqq{}_{cs}A\cdot{}_{cs}F_c+{}_{mw}A\cdot{}_{w}\sigma_Y+{}_{w}t\cdot{}_{w}l'\cdot F_c$	$(N_U-{}_{cs}A\cdot{}_{cs}F_c+{}_{mw}A\cdot{}_{w}\sigma_Y)$ $\times\left(1-\frac{N_U-{}_{cs}A\cdot{}_{cs}F_c+{}_{mw}A\cdot{}_{w}\sigma_Y}{{}_{w}t\cdot{}_{w}l'\cdot F_c}\right)\frac{{}_{w}l'}{2}$ $+\left({}_{scs}A\cdot{}_{s}\sigma_Y+{}_{mcs}A\cdot{}_{m}\sigma_Y+\frac{{}_{cs}A\cdot{}_{cs}F_c}{2}\right){}_{w}l$
${}_{cs}A\cdot{}_{cs}F_c+{}_{mw}A\cdot{}_{w}\sigma_Y+{}_{w}t\cdot{}_{w}l'\cdot F_c<N_U$ $\leqq2({}_{cs}A\cdot{}_{cs}F_c+{}_{mcs}A\cdot{}_{m}\sigma_Y+{}_{scs}A\cdot{}_{s}\sigma_Y)$ $+{}_{mw}A\cdot{}_{w}\sigma_Y+{}_{w}t\cdot{}_{w}l'\cdot F_c$	$\left(-\frac{N_U}{2}+{}_{cs}A\cdot{}_{cs}F_c+{}_{mcs}A\cdot{}_{m}\sigma_Y+{}_{scs}A\cdot{}_{s}\sigma_Y\right){}_{W}l$ $+({}_{mw}A\cdot{}_{m}\sigma_Y+{}_{w}t\cdot{}_{w}l'\cdot F_c)\frac{{}_{w}l}{2}$

Appendix D Allowable Flexural Strength of Composite Steel Tube and Concrete Columns

Table D1 Allowable Flexural Strength of Circular-hollow Reinforced Concrete Portion Covering Steel Tube

x_{n1}	${}_rN/(bD\cdot f_c')$, ${}_rM/(bD^2\cdot f_c')$	
$0\leqq x_{n1}<{}_sd_1$	${}_rN/(bD\cdot f_c')$	$x_{n1}/2+n\cdot{}_mp_t(2-1/x_{n1})$
	${}_rM/(bD^2\cdot f_c')$	$(3-2x_{n1})x_{n1}/12+n\cdot{}_mp_t(1-2{}_rd_{t1})^2/(2x_{n1})$
${}_sd_1\leqq x_{n1}<1-{}_sd_1$	${}_rN/(bD\cdot f_c')$	$x_{n1}/2+n\cdot{}_mp_t(2-1/x_{n1})-\eta(1-2{}_sd_1)^3$ $\times\left\{\frac{\sin\theta_n}{3}(2+\cos^2\theta_n)-\theta_n\cos\theta_n\right\}\Big/(8x_{n1})$
	${}_rM/(bD^2\cdot f_c')$	$(3-2x_{n1})x_{n1}/12+n\cdot{}_mp_t(1-2{}_rd_{t1})^2/(2x_{n1})$ $-\eta(1-2{}_sd_1)^4\{3\theta_n+\sin2\theta_n(\cos^2\theta_n-5/2)\}/(192x_{n1})$
$1-{}_sd_1\leqq x_{n1}<1$	${}_rN/(bD\cdot f_c')$	$x_{n1}/2+n\cdot{}_mp_t(2-1/x_{n1})-\pi\cdot\eta(1-2{}_sd_1)^2\{1-1/(2x_{n1})\}/4$
	${}_rM/(bD^2\cdot f_c')$	$(3-2x_{n1})x_{n1}/12+n\cdot{}_mp_t(1-2{}_rd_{t1})^2/(2x_{n1})-\pi\cdot\eta(1-2{}_sd_1)^4/(64x_{n1})$
$1\leqq x_{n1}$	${}_rN/(bD\cdot f_c')$	$\{1+2n\cdot{}_mp_t-\pi\cdot\eta(1-2{}_sd_1)^2/4\}\cdot\{1-1/(2x_{n1})\}$
	${}_rM/(bD^2\cdot f_c')$	$\{1/12+n\cdot{}_mp_t(1-2{}_rd_{t1})^2/2-\pi\cdot\eta(1-2{}_sd_1)^4/64\}/x_{n1}$

$\theta_n=\cos^{-1}\left(\frac{1-2x_{n1}}{1-2{}_sd_1}\right)$, $\eta=D/b$, ${}_rd_{t1}={}_rd_t/D$, ${}_sd_1=(D-{}_sD)/(2D)$, $x_{n1}=x_n/D$
$f_c'=f_c(1-4{}_sp)$

Table D2 Allowable Flexural Strength of Square-hollow Reinforced Concrete Portion Covering Steel Tube

x_{n1}	${}_rN/(bD\cdot f_c')$, ${}_rM/(bD^2\cdot f_c')$	
$0\leqq x_{n1}<{}_sd_1$	${}_rN/(bD\cdot f_c')$	$x_{n1}/2+n\cdot{}_mp_t(2-1/x_{n1})$
	${}_rM/(bD^2\cdot f_c')$	$(3-2x_{n1})x_{n1}/12+n\cdot{}_mp_t(1-2{}_rd_{t1})^2/(2x_{n1})$
${}_sd_1\leqq x_{n1}<1-{}_sd_1$	${}_rN/(bD\cdot f_c')$	$x_{n1}/2+n\cdot{}_mp_t(2-1/x_{n1})-\eta(1-2{}_sd_1)(x_{n1}-{}_sd_1)^2/(2x_{n1})$
	${}_rM/(bD^2\cdot f_c')$	$(3-2x_{n1})x_{n1}/12+n\cdot{}_mp_t(1-2{}_rd_{t1})^2/(2x_{n1})$ $-\eta(1-2{}_sd_1)(x_{n1}-{}_sd_1)^2\{1/2-(x_{n1}+2{}_sd_1)/3\}/(2x_{n1})$
$1-{}_sd_1\leqq x_{n1}<1$	${}_rN/(bD\cdot f_c')$	$x_{n1}/2+n\cdot{}_mp_t(2-1/x_{n1})-\eta(1-2{}_sd_1)^2\{1-1/(2x_{n1})\}$
	${}_rM/(bD^2\cdot f_c')$	$(3-2x_{n1})x_{n1}/12+n\cdot{}_mp_t(1-2{}_rd_{t1})^2/(2x_{n1})-\eta(1-2{}_sd_1)^4/(12x_{n1})$
$1\leqq x_{n1}$	${}_rN/(bD\cdot f_c')$	$\{1+2n\cdot{}_mp_t-\eta(1-2{}_sd_1)^2\}\ \{1-1/(2x_{n1})\}$
	${}_rM/(bD^2\cdot f_c')$	$\{1/12+n\cdot{}_mp_t(1-2{}_rd_{t1})^2/2-\eta(1-2{}_sd_1)^4/12\}/x_{n1}$

Table D3 Allowable Flexural Strength of Filled Concrete Portion

	x_{n1}	${}_cN/({}_cD^2\cdot f_c')$, ${}_cM/({}_cD^3\cdot f_c')$	
Circular tube	Inside section	${}_cN/({}_cD^2\cdot f_c')$	$\{\sin\theta_n(2+\cos^2\theta_n)/3-\theta_n\cos\theta_n\}/(8x_{n1})$*
		${}_cM/({}_cD^3\cdot f_c')$	$\{\theta_n+\sin2\theta_n(\cos^2\theta_n-5/2)/3\}/(64x_{n1})$*
	Outside section ($e\leqq{}_cD/8$)	${}_cN/({}_cD^2\cdot f_c')$	$\pi\{1-1/(2x_{n1})\}/4$
		${}_cM/({}_cD^3\cdot f_c')$	$\pi/(64x_{n1})$
Square tube with constant thickness	Inside section	${}_cN/({}_cD^2\cdot f_c')$	$x_{n1}/2$
		${}_cM/({}_cD^3\cdot f_c')$	$x_{n1}(3-2x_{n1})/12$
	Outside section ($e\leqq{}_cD/6$)	${}_cN/({}_cD^2\cdot f_c')$	$1-1/(2x_{n1})$
		${}_cM/({}_cD^3\cdot f_c')$	$1/(12x_{n1})$

* $\theta_n=\cos^{-1}(1-2x_{n1})$, $x_{n1}=x_n/{}_cD$, $f_c'=f_c$

Appendix E Ultimate Flexural Strength of Symmetrical Hollow Concrete Section

Table E1 Ultimate Flexural Strength of Symmetrical Hollow Concrete Section

	x_{n1}	${}_cN_U/(bD\cdot F_c)$	${}_cM_U/(bD^2\cdot F_c)$
Square-hollow rectangular Section	$0\leqq x_{n1}<{}_sd_1$	—	$\dfrac{1}{2}\dfrac{{}_cN_U}{bD\cdot F_c}\left(1-\dfrac{1}{{}_c\gamma_U}\dfrac{{}_cN_U}{bD\cdot F_c}\right)$
	${}_sd_1\leqq x_{n1}<1-{}_sd_1$	${}_c\gamma_U\left\{x_{n1}-\dfrac{D}{b}(1-2{}_sd_1)\times(x_{n1}-{}_sd_1)\right\}$	${}_c\gamma_U\left\{\dfrac{x_{n1}(1-x_{n1})}{2}-\dfrac{D}{b}\times\dfrac{(1-2{}_sd_1)(x_{n1}-{}_sd_1)(1-x_{n1}-{}_sd_1)}{2}\right\}$
	$1-{}_sd_1\leqq x_{n1}\leqq 1$	${}_c\gamma_U\left\{x_{n1}-\dfrac{D}{b}(1-2{}_sd_1)^2\right\}$	${}_c\gamma_U\dfrac{x_{n1}(1-x_{n1})}{2}$
Circular-hollow rectangular section	$0\leqq x_{n1}<{}_sd_1$	—	$\dfrac{1}{2}\dfrac{{}_cN_U}{bD\cdot F_c}\left(1-\dfrac{1}{{}_c\gamma_U}\dfrac{{}_cN_U}{bD\cdot F_c}\right)$
	${}_sd_1\leqq x_{n1}<1-{}_sd_1$	${}_c\gamma_U\left\{x_{n1}-\dfrac{D}{b}\dfrac{(1-{}_sd_1)^2}{4}\cdot\times(\theta_n-\sin\theta_n\cdot\cos\theta_n)\right\}$	${}_c\gamma_U\left\{\dfrac{x_{n1}(1-x_{n1})}{2}-\dfrac{D}{b}\dfrac{(1-2{}_sd_1)^3}{12}\sin^3\theta_n\right\}$
	$1-{}_sd_1\leqq x_{n1}\leqq 1$	${}_c\gamma_U\left\{x_{n1}-\dfrac{D}{b}\cdot\dfrac{\pi(1-2{}_sd_1)^2}{4}\right\}$	${}_c\gamma_U\dfrac{x_{n1}(1-x_{n1})}{2}$

$\theta_n=\cos^{-1}\left(\dfrac{1-2x_{n1}}{1-2{}_sd_1}\right)$

Appendix F Notation

A_e : effective area of concrete (cm^2) (Article 17)
A_s : area of stiffener (cm^2) (20)
${}_aA$: area of anchor bolt (cm^2) (Appendix A)
${}_{cs}A$: area of column connected to structural wall (cm^2) (C)
${}_cA$: area of concrete portion (cm^2) (40, 43)
${}_DA$: area of lattice (cm^2) (18, 33), area of brace (cm^2) (25, 37)
${}_mA$: total area of longitudinal reinforcement in column (cm^2) (17, A)
${}_{mcs}A$: total area of longitudinal reinforcement in column adjacent to structural wall (cm^2) (C)
${}_{mw}A$: total area of vertical reinforcement in structural wall (cm^2) (C)
${}_sA$: area of steel portion (cm^2) (17, 41, 43, 46, B)
${}_{scs}A$: area of steel portion in column adjacent to structural wall (cm^2) (C)
${}_sA_e$: net area of steel portion deducting bolt holes (cm^2) (17)
${}_ma_c$: area of compression reinforcement (cm^2) (32)
${}_ma_m$: area of intermediate reinforcement (cm^2) (B)
${}_ma_t$: area of tension reinforcement (cm^2) (16, 19, 32, A, B)
${}_sa_f$: area of steel flange (cm^2) (B)
${}_sa_m$: area of intermediate steel (cm^2) (B)
${}_sa_t$: net area of tension chord of steel deducting bolt holes (cm^2) (16, 17)
${}_sa_w$: area of steel web (cm^2) (32, B)
${}_wa$: area of a pair of stirrups or hoops (cm^2) (7, 22, 35)
B : effective width of T-section (cm) (16)
b : width of rectangular section or web of T-section (cm) (7, 16, 17, 18, 22, 33, 36, 40, A, B, D)
b' : effective width of concrete at steel flange (cm) (7, 18, 33, 41, 46)
b_e : effective width concerning bearing strength of steel in embedded type column base (cm) (22, 35, 44)
b_f : width of steel flange (cm) (18, 33, B)
${}_Bb$: width of beam (cm) (20, 43)
${}_bb$: width of base plate (cm) (A)
${}_cb$: width of column (cm) (20, 43)
D : depth of flexural member (cm) (17, 40, 41, A, B, D, E)

D_h : diameter of opening in beam (cm) (18)
${}_bD$: depth of concrete below base plate (cm) (A)
${}_cD$: depth or diameter of filled concrete (cm) (D)
${}_sD$: diameter of steel tube (cm) (43, 44, B, D)
d_f : distance between flange surface and toe of web fillet (cm) (20, 22)
d_w : depth of steel web (cm) (18, 33)
${}_ad$: center distance between tension and compression anchor bolts (cm) (A)
${}_md$: center distance between tension and compression reinforcement (cm) (32, A, B)
${}_{mB}d$: center distance between upper and lower reinforcement in beam (cm) (20, 43)
${}_{mC}d$: center distance between right and left reinforcement in column (cm) (20, 43)
${}_rd$: distance between extreme compression fiber and center of tension reinforcement (cm) (16, 18)
${}_rd_c$: distance between extreme compression fiber and center of compression reinforcement (cm) (16)
${}_rd_{c1}$: ${}_rd_c/D$ (17, 40)
${}_rd_t$: distance between extreme tension fiber and center of tension reinforcement (cm) (40, D)
${}_rd_{t1}$: ${}_rd_t/D$ (17, 40, D)
${}_sd$: center distance between tension and compression chords of flanges of steel (cm) (16, 17, 18)
${}_sd_1$: ratio of cover concrete for steel to total depth (D, E)
${}_{sB}d$: center distance between upper and lower chords or flanges of steel in beam (cm) (20, 43)
${}_{sC}d$: center distance between right and left chords or flanges of steel in column (cm) (20)
${}_cE$: modulus of elasticity of concrete (kg/cm^2) (17, 40)
${}_sE$: modulus of elasticity of steel (kg/cm^2) (17)
F : standard value to determine allowable stresses of steel (kg/cm^2) (14)
F_B : bearing strength of concrete (kg/cm^2) (35)
F_c : design standard strength of concrete (kg/cm^2) (9, 10, 14, 17, 20, 33, 36, 37, B, C)
F_s : shear strength of concrete (kg/cm^2) (33, 46)
${}_jF_s$: shear strength of concrete in beam-to-column connection (kg/cm^2) (36)
${}_{cs}F_c$: compressive strength of concrete in column adjacent to structural wall, taken equal to $0.75F_c$ (kg/cm^2) (37, C)
${}_wF_s$: shear cracking strength of concrete in structural wall (kg/cm^2) (37)
${}_wF_s'$: shear strength provided by compression field of concrete in structural wall (kg/cm^2) (37)
f_c : allowable compressive stress of concrete (kg/cm^2) (16, 17, 22, 25, 40, A)

f_c' : allowable compressive stress used in calculation for RC portion of column (kg/cm^2) (17, 22, 40, A, D)
f_B : allowable bearing stress of concrete (kg/cm^2) (22)
f_s : allowable shear stress of concrete (kg/cm^2) (18, 20, 25, 41, 43)
f_s' : allowable shear stress for compression field of concrete in structural wall (kg/cm^2) (25)
${}_af_t$: allowable tensile stress of anchor bolt (kg/cm^2) (A)
${}_mf_a$: allowable bond stress of longitudinal reinforcement (kg/cm^2) (19)
${}_mf_c$: allowable compressive stress of longitudinal reinforcement (kg/cm^2) (17, A)
${}_mf_t$: allowable tensile stress of longitudinal reinforcement (kg/cm^2) (16, 17, 40, A)
${}_sf_b$: allowable flexural stress of steel (kg/cm^2) (16)
${}_sf_c$: allowable compressive stress of steel (kg/cm^2) (17)
${}_sf_c'$: allowable stress of steel for local compression (kg/cm^2) (20)
${}_sf_s$: allowable shear stress of steel (kg/cm^2) (18, 20, 25, 41, 43)
${}_sf_t$: allowable tensile stress of steel (kg/cm^2) (16, 17, 18, 20, 25)
${}_wf_t$: allowable tensile stress of wall reinforcement, taken equal to ${}_mf_t$ (kg/cm^2) (25)
${}_wf_t$: allowable tensile stress of stirrups or hoops used for shear design (kg/cm^2) (18, 20, 22, 41)
h : story height (cm) (20, 25, 43)
h' : clear height of column (cm) (18, 20, 43)
h_h : height of opening in structural wall (cm) (25)
${}_bh$: embedded length of steel column base (cm) (22, 35)
I_n : moment of inertia of effective equivalent cross section about neutral axis, which neglects concrete section in tension (cm^4) (40)
${}_cI$: moment of inertia of concrete portion (cm^4) (17, 40)
${}_sI$: moment of inertia of steel portion (cm^4) (17)
${}_rj$: distance between centroids of tension and compression in RC portion under flexure, taken equal to $(7/8)_rd$ (cm) (16, 18, 19, 33, 41, 46)
l : span length of beam (cm) (25, 37)
l' : clear span length of beam (cm) (18, 21, 33, 46)
l_d : development length of reinforcing bar for bond strength (cm) (19)
l_h : length of openting in structural wall (cm) (25)
l_j : distance between steel joint and member end (cm) (21)
l_k : buckling length of column (cm) (17, 40)
${}_wl$: length of structural wall (cm) (C)
${}_wl'$: clear length of structural wall (cm) (C)
M : design bending moment (kg·cm) (16, 17, 18, 21, 22, 40)

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M_{BA} : allowable flexural strength of batten-plate type steel chord (kg·cm) (18)
M_{BU} : ultimate flexural strength of batten-plate type steel chord (kg·cm) (33)
M_{Ej} : bending moment at steel joint caused by earthquake load (kg·cm) (21)
M_j : bending moment at steel joint (kg·cm) (21)
M_{j0} : bending moment at steel joint of simply-supported steel beam due to intermediate load (kg·cm) (21)
M_{Lj} : bending moment due to long-term load at steel joint (kg·cm) (21)
M_U : ultimate flexural strength of member (kg·cm) (32)
M_x : design bending moment about x-axis (kg·cm) (17, 40)
M_y : design bending moment about y-axis (kg·cm) (17, 18, 40)
${}_aM$: bending moment carried by anchor bolt (kg·cm) (22, A)
${}_aM_o$: allowable flexural strength of anchor bolt under axial force equal to ${}_aN_t/2$ (kg·cm) (22, A)
${}_BM_1$: bending moment in beam-to-column connection transmitted from beam 1 (kg·cm) (20, 43)
${}_BM_2$: bending moment in beam-to-column connection transmitted from beam 2 (kg·cm) (20, 43)
${}_{BS}M_A$: allowable flexural strength of beam adjacent to structural wall (kg·cm) (25)
${}_{BS}M_U$: ultimate flexural strength of beam adjacent to structural wall (kg·cm) (37)
${}_{BT}M_A$: allowable flexural strength of beam with sagging wall or panel wall (kg·cm) (25)
${}_{BT}M_U$: ultimate flexural strength of beam with sagging wall or panel wall (kg·cm) (37)
${}_bM$: bending moment carried by concrete underneath base plate (kg·cm) (22, A)
${}_{CS}M_A$: allowable flexural strength of column adjacent to structural wall (kg·cm) (25)
${}_{CS}M_U$: ultimate flexural strength of column adjacent to structural wall (kg·cm) (37)
${}_{CT}M_A$: allowable flexural strength of column with side wall (kg·cm) (25)
${}_{CT}M_U$: ultimate flexural strength of column with side wall (kg·cm) (37)
${}_cM$: allowable flexural strength of concrete portion (kg·cm) (22, 40, A, E)
${}_cM_U$: ultimate flexural strength of filled concrete portion (kg·cm) (32, 35, 45, 46, B)
${}_cM_x$: allowable flexural strength about x-axis of filled concrete portion (kg·m) (40)
${}_cM_y$: allowable flexural strength about y-axis of filled concrete portion (kg·cm) (40)
${}_JM_U$: ultimate shear strength of beam-to-column connection converted to bending moment (kg·cm) (36, 47)
${}_mM$: allowable flexural strength of longitudinal reinforcement (kg·cm) (22, A)
${}_mM_o$: allowable flexural strength of longitudinal reinforcement subjected to bending alone (kg·cm) (22, A)
${}_mM_U$: ultimate flexural strength of longitudinal reinforcement (kg·cm) (32, 35, B)
${}_mM_{U0}$: ultimate flexural strength of longitudinal reinforcement subjected to bending alone

(kg·cm) (32)
${}_rM$: allowable flexural strength of RC portion (kg·cm) (17, 22, 40, D)
${}_rM_o$: allowable flexural strength of RC portion subjected to bending alone (kg·cm) (16, 17)
${}_rM_1$: absolute value of ultimate flexural strength of RC portion at member end 1 (kg·cm) (18, 41)
${}_rM_2$: absolute value of ultimate flexural strength of RC portion at member end 2 (kg·cm) (18, 41)
${}_rM_b$: balanced flexural strength of RC portion (kg·cm) (40)
${}_rM_d$: design bending moment for RC portion (kg·cm) (18)
${}_rM_U$: ultimate flexural strength of RC portion (kg·cm) (32, 33, 35, 46, B)
${}_rM_x$: allowable flexural strength of RC portion about x-axis (kg·cm) (17, 40)
${}_rM_y$: allowable flexural strength of RC portion about y-axis (kg·cm) (17, 40)
${}_{rB}M_A$: allowable flexural strength of RC portion in beam (kg·cm) (20)
${}_{rC}M_A$: allowable flexural strength of RC portion in column (kg·cm) (20)
${}_sM$: allowable flexural strength of steel portion (kg·cm) (17, 40), allowable flexural strength of the portion underneath base plate at column base treated as steel (kg·cm) (22)
${}_sM_o$: allowable flexural strength of steel portion subjected to bending alone (kg·cm) (16, 17, 40)
${}_sM_1$: absolute value of allowable flexural strength of steel portion at member end 1 under short-term loading condition (kg·cm) (21)
${}_sM_2$: absolute value of allowable flexural strength of steel portion at member end 2 under short-term loading condition (kg·cm) (21)
${}_sM_d$: design bending moment for steel portion (kg·cm) (18, 21)
${}_sM_j$: design bending moment for steel joint (kg·cm) (21)
${}_sM_U$: ultimate flexural strength of steel portion (kg·cm) (32, 33, 35, 45, B)
${}_sM_{U0}$: ultimate flexural strength of steel portion subjected to bending alone (kg·cm) (32)
${}_sM_{U1}$: ultimate flexural strength of steel portion of column base (kg·cm) (35)
${}_sM_{U2}$: ultimate flexural strength provided by side pressure to embedded steel portion of column base (kg·cm) (35)
${}_sM_{U3}$: ultimate flexural strength of the portion underneath base plate of embedded type column base (kg·cm) (35)
${}_sM_x$: allowable flexural strength about x-axis of steel portion (kg·cm) (17, 40)
${}_sM_y$: allowable flexural strength about y-axis of steel portion (kg·cm) (17, 40)
${}_{sB}M_A$: allowable flexural strength of steel portion in beam (kg·cm) (20)
${}_{sC}M_A$: allowable flexural strength of steel portion in column (kg·cm) (20)
${}_rm$: dimensionless allowable flexural strength of RC portion (18)

N : design compressive force (kg) (17, 18, 37, 40, 41, 46, C)
N_k : buckling strength of column (kg) (17, 40)
N_l : limiting value for axial force of column (kg) (17)
N_U : ultimate compressive strength of member (kg) (32, B)
${}_aN$: allowable tensile strength of anchor bolt, taken negative (kg) (22, A)
${}_aN_t$: allowable tensile strength of anchor bolt subjected to tension alone, taken negative (kg) (22, A)
${}_bN$: allowable compressive strength of the portion underneath base plate (kg) (22, A)
${}_bN_c$: allowable compressive strength of the portion underneath base plate subjected to compression alone (kg) (22, A)
${}_cN$: allowable compressive strength of concrete portion (kg) (22, 40, 41, A)
${}_cN_c$: allowable compressive strength of concrete portion subjected to compression alone (kg) (22, 40, 41, 41, A)
${}_cN_{cU}$: ultimate compressive strength of concrete portion subjected to compression alone (kg) (32)
${}_cN_k$: buckling strength of concrete portion (kg) (40)
${}_cN_U$: ultimate compressive strength of concrete portion (kg) (32, B, E)
${}_mN$: allowable compressive strength of longitudinal reinforcement (kg) (22, A)
${}_mN_c$: allowable compressive strength of longitudinal reinforcement subjected to compression alone (kg) (22, A)
${}_mN_t$: allowable tensile strength of longitudinal reinforcement subjected to tension alone, taken negative (kg) (22, A)
${}_mN_U$: ultimate compressive strength of longituinal reinforcement (kg) (32, B)
${}_rN$: allowable compressive strength of RC portion (kg) (17, 22, 40, D)
${}_rN_b$: balanced compressive strength of RC portion (kg) (40, 41)
${}_rN_c$: allowable compressive strength of RC portion subjected to compression alone (kg) (17, 18, 40, 41)
${}_rN_{c1}$: allowable compressive strength of RC portion subjected to compression alone, controlled by allowable compressive stress of concrete (kg) (17)
${}_rN_{c2}$: allowable compressive strength of RC portion subjected to compression alone, controlled by allowable compressive stress of longituinal reinforcement (kg) (17)
${}_rN_{cU}$: ultimate compressive strength of RC portion subjected to compression alone (kg) (32)
${}_rN_k$: buckling strength of RC portion (kg) (17)
${}_rN_t$: allowable tensile strength of RC portion subjected to tension alone, taken negative (kg) (17, 40)
${}_rN_{tU}$: ultimate tensile strength of RC portion subjected to tension alone, taken negative (kg) (32)

${}_rN_U$: ultimate compressive strength of RC portion (kg) (32, 35, B)
${}_sN$: allowable compressive strength of steel portion (kg) (17, 21, 40), allowable compressive strength of the portion underneath base plate at column base treated as steel (kg) (22)
${}_sN_c$: allowable compressive strength of steel portion subjected to compression alone (kg) (17, 40)
${}_sN_j$: axial force in steel joint (kg) (21)
${}_sN_t$: allowable tensile strength of steel portion subjected to tension alone, taken negative (kg) (17)
${}_sN_U$: ultimate compressive strength of steel portion (kg) (32, 35, B)
n : ratio of moduli of elasticity of steel and concrete (16, 17, 40, D)
${}_rn$: dimensionless allowable axial strength of RC portion (17)
P : axial force in stiffener of beam-to-column connection (kg) (20)
${}_rp_c$: compression reinforcement ratio (40)
${}_mp_t$: tension reinforcement ratio (16, 17, 40, D)
${}_mp_{tb}$: balanced reinforcement ratio of beam (16)
${}_sp$: steel tube ratio (${}_sA/bD$) (40, 45)
${}_sp_c$: compression steel ratio (${}_sa_c/bD$) (17, 32, B)
${}_wp$: wall reinforcement ratio (25, 37)
${}_wp$: stirrup ratio or hoop ratio (7, 18, 20, 33, 36, 41, 46)
${}_wp_e$: effective stirrup or hoop ratio (18)
${}_wp_h$: web reinforcement ratio of beam with opening (18)
Q : design shear force (kg) (18, 19, 25)
Q_o : shear force due to intermediate load of simply-supported beam (kg) (18, 21)
Q_A : allowable shear strength of steel reinforced concrete column under long-term loading condition, or allowable shear strength for the design of steel reinforced concrete beam with batten-plate type steel treated as equivalent RC beam (kg) (18, 41)
Q_{A1} : allowable shear strength in Q_A controlled by shear failure (kg) (18)
Q_{A2} : allowable shear strength in Q_A controlled by shear bond failure (kg) (18)
Q_{AS} : allowable shear strength under short-term loading condition for the design of SRC beam with batten-plate type steel treated as equivalent RC beam (kg) (18)
Q_{AS1} : allowable shear strength in Q_{AS} controlled by shear failure (kg) (18)
Q_{AS2} : allowable shear strength in Q_{AS} controlled by shear bond failure (kg) (18)
Q_E : shear force caused by earthquake load (kg) (18)
Q_{Ej} : shear force at steel joint caused by earthquake load (kg) (21)
Q_j : design shear force for steel joint (kg) (21)
Q_{jo} : shear force at joint of simply-supported steel beam due to intermediate load (kg) (21)

Q_{Lj} : shear force at steel joint due to long-term load (kg) (21)

Q_U : ultimate shear strength of member (kg) (33, 46)

${}_{BS}Q_A$: allowable shear strength of beam adjacent to structural wall (kg) (25)

${}_{BS}Q_U$: ultimate shear strength of beam adjacent to structural wall (kg) (37)

${}_{BT}Q_A$: allowable shear strength of beam with sagging wall or panel wall (kg) (25)

${}_{BT}Q_U$: ultimate shear strength of beam with sagging wall or panel wall (kg) (37)

${}_{c}Q_U$: ultimate shear strength of concrete portion (kg) (46)

${}_{CS}Q_A$: allowable shear strength of column adjacent to structural wall (kg) (25)

${}_{CS}Q_U$: ultimate shear strength of column adjacent to structural wall (kg) (37)

${}_{CT}Q_A$: allowable shear stengthe of column with side wall (kg) (25)

${}_{CT}Q_U$: ultimate shear strength of column with side wall (kg) (37)

${}_{r}Q_A$: allowable shear strength of RC portion in beam (kg) (18)

${}_{r}Q_{A1}$: allowable shear strength in ${}_{r}Q_A$ controlled by shear failure (kg) (18)

${}_{r}Q_{A2}$: allowable shear strength in ${}_{r}Q_A$ controlled by shear bond failure (kg) (18)

${}_{r}Q_{AL}$: allowable shear strength under long-term loading condition of RC portion in column (kg) (18, 41)

${}_{r}Q_{AS}$: allowable shear strength under short-term loading condition of RC portion in column (kg) (18, 41)

${}_{r}Q_{AS1}$: allowable shear strength in ${}_{r}Q_{AS}$ controlled by shear failure (kg) (18)

${}_{r}Q_{AS2}$: allowable shear strength in ${}_{r}Q_{AS}$ controlled by shear bond failure (kg) (18)

${}_{r}Q_{bU}$: ultimate shear strength in ${}_{r}Q_U$ controlled by flexural failure of RC portion (kg) (33)

${}_{r}Q_d$: design shear force for RC portion (kg) (18, 19, 41)

${}_{r}Q_{sU}$: ultimate shear strength in ${}_{r}Q_U$ controlled by shear of RC portion (kg) (33, 46)

${}_{r}Q_{sU1}$: ultimate shear strength in ${}_{r}Q_{sU}$ controlled by shear failure (kg) (33)

${}_{r}Q_{sU2}$: ultimate shear strength in ${}_{r}Q_{sU}$ controlled by shear bond failure (kg) (33)

${}_{r}Q_U$: ultimate shear strength of RC portion (kg) (33, 46)

${}_{s}Q_A$: allowable shear strength of steel portion (kg) (18, 25, 41)

${}_{s}Q_{bU}$: ultimate shear strength in ${}_{s}Q_U$ controlled by flexural yielding of steel portion (kg) (33)

${}_{s}Q_d$: design shear force for steel portion (kg) (18, 22, 41)

${}_{s}Q_j$: design shear force for steel joint (kg) (21)

${}_{s}Q_{sU}$: ultimate shear strength in ${}_{s}Q_U$ controlled by shear failure of steel portion (kg) (33, 46)

${}_{s}Q_U$: ultimate shear strength of steel portion (kg) (33, 35, 37, 46)

${}_{w}Q_A$: allowable shear strength under short-term loading condition of structural wall (kg) (25)

${}_{w}Q_{A1}$: allowable shear strength in ${}_{w}Q_A$ controlled by diagonal tension cracking of wall (kg) (25)

${}_{w}Q_{A2}$: allowable shear strength in ${}_{w}Q_A$ controlled by strength attained after diagonal tension cracking of wall (kg) (25)

${}_{w}Q'_{A1}$: allowable shear strength of structural wall without large opening (kg) (25)

${}_{w}Q'_{A2}$: allowable shear strength of structural wall with large opening (kg) (25)

${}_{w}Q_U$: ultimate shear strength of structural wall (kg) (37)

${}_{w}Q_{U1}$: ultimate shear strength in ${}_{w}Q_U$ controlled by diagonal tension cracking of wall (kg) (37)

${}_{w}Q_{U2}$: ultimate shear strength in ${}_{w}Q_U$ controlled by strength attained after diagonal tension cracking of wall (kg) (37)

${}_{w}Q'_{U1}$: ultimate shear strength of structural wall without large opening (kg) (37)

${}_{w}Q'_{U2}$: ultimate shear strength of structural wall with large opening (kg) (37)

${}_{BT}R_A$: allowable strength of beam with sagging wall or panel wall (kg) (25)

${}_{BT}R_U$: ultimate strength of beam with sagging wall or panel wall (kg) (37)

${}_{CT}R_A$: allowable strength of column with side wall (kg) (25)

${}_{CT}R_U$: ultimate strength of column with side wall (kg) (37)

r : strength reduction factor for structural wall with opening (25, 37)

r_1 : factor in r determined by ratio of opening length to wall length (25)

r_2 : factor in r determined by ratio of opening area to wall area (25)

r_3 : factor in r determined by ratio of opening height to wall height (25)

r_h : reduction factor for allowable shear strength of steel portion of beam with opening (18)

${}_{c}r_U$: reduction factor for F_c of concrete determined by steel ratio (32, 35, 45, B, E)

S_n : first moment of area about neutral axis of effective equivalent section (cm^3) (40)

T_B : allowable tension strength of batten plate (kg) (18)

T_h : edge tension force acting around opening of structural wall (kg) (25)

${}_{BS}T$: tension force caused by earthquake load in beam adjacent to structural wall (kg) (25)

${}_{BS}T_U$: tension force caused at ultimate strength state in beam adjacent to structural wall (kg) (37)

${}_{CS}T$: tension force caused by earthquake load in column adjacent to structural wall (kg) (25)

${}_{CS}T_U$: tension force caused at ultimate strength state in column adjacent to structural wall (kg) (37)

t_1 : ratio of slab thickness to effective depth of beam (16)

t_f : thickness of steel flange (cm) (18, 33)

t_w : web thickness of full-web steel (cm) (18, 22, 33)

${}_{B}t_f$: flange thickness of steel beam (cm) (20)

${}_{c}t_w$: web thickness of steel column (cm) (20)

${}_{j}t_w$: web thickness of steel beam-to-column connection (cm) (20)

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${}_{s}t$: wall thickness of steel tube (cm) (44)
${}_{sW}t$: thickness of steel structural wall (cm) (25, 37)
${}_{W}t$: thickness of structural wall (cm) (25, 37, C)
${}_{c}V$: volume of concrete portion of beam-to-column connection (cm^3) (20, 42, 47)
${}_{c}V_e$: volume of effective concrete portion of beam-to-column connection (cm^3) (20, 36, 43, 47)
${}_{s}V$: volume of steel web of beam-to-column connection (cm^3) (36, 43, 47)
x : spacing of stirrups or hoops (cm) (7, 22, 35)
x_B : spacing of batten plates (cm) (18, 33)
x_n : distance between extreme compression fiber and neutral axis (cm) (17, A, D, E)
x_{n1} : neutral axis ratio of RC portion of column (x_n/D) (17, A, D, E)
x_{n1b} : neutral axis ratio at balanced state of RC portion of column (40)
Y : yield ratio of steel (21)
${}_{s}Z$: section modulus of steel portion (cm^3) (16, 17)
${}_{s}Z_p$: plastic section modulus of steel portion (cm^3) (32, B)
α : coefficient related to shear span ratio ${}_{r}M/({}_{r}Q \cdot {}_{r}d)$ of member (18, 41, 46)
α' : coefficient determined by α and b'/b (18)
${}_{r}\alpha$: coefficient related to shear span ratio $M/({}_{r}Q \cdot d)$ of RC portion (18)
β : coefficient related to type and dimension of steel web (18, 19, 20, 25, 37, 41, 43)
${}_{BS}\beta_b$: coefficient related to flexural strength determined according to stress state of beam adjacent to structural wall (25, 37)
${}_{BS}\beta_s$: coefficient related to shear strength determined according to stress state of beam adjacent to structural wall (25, 37)
${}_{CS}\beta_b$: coefficient related to flexural strength determined according to stress state of column adjacent to structural wall (25, 37)
${}_{CS}\beta_s$: coefficient related to shear strength determined according to stress state of column adjacent to structural wall (25, 37)
${}_{J}\beta$: coefficient related to shape of beam-to-colum connection with concrete filled steel tubular column (43, 47)
γ : ratio of compression reinforcements to tension reinforcements (${}_{m}a_c/{}_{m}a_t$) (16)
${}_{c}\gamma$: weight of concrete per unit volume (t/m^3) (10)
${}_{J}\delta$: coefficient related to shape of beam-to-column connection (20, 36, 43, 47)
${}_{c}\delta$: moment magnification factor for concrete portion of long column (40)
${}_{r}\delta$: moment magnification factor for RC portion of long column (17)
η : ratio of depth to width of rectangular cross section (D)
θ_n : angle related to location of neutral axis in circular cross section (B, D, E)
${}_{D}\theta$: angle between lattice and member axis, or angle between brace and horizontal axis (18, 25, 33, 37)

${}_{w}\theta$: angle between reinforcement for opening and axis of beam (18)
${}_{r}\mu$: coefficient multiplied to compressive strength of concrete portion determining limit to axial force in column (17)
${}_{s}\mu$: coefficient multiplied to compressive strength of steel portion determining limit to axial force in column (17)
ν_j : stress magnification factor for design of steel joint (21)
${}_{c}\nu$: factor of safety for buckling of concrete portion (40)
${}_{r}\nu$: factor of safety for buckling of RC portion (17)
${}_{s}\nu$: factor of safety for buckling of steel portion (17)
${}_{m}\sigma_d$: tensile stress in tension reinforcement of flexural member, may be reduced to 2/3 for check of bond stress when bar is hooked at end (kg/cm^2) (19)
${}_{m}\sigma_Y$: yield stress of longitudinal reinforcement (kg/cm^2) (32, B, C)
${}_{s}\sigma_Y$: yield stress of steel (kg/cm^2) (32, 33, 36, 37, 46, 47, B, C)
${}_{w}\sigma_Y$: yield stress of web reinforcement (kg/cm^2) (33, 36, 46)
${}_{W}\sigma_Y$: yield stress of wall reinforcement (kg/cm^2) (37, B, C)
${}_{m}\tau_a$: bond stress of longitudinal reinforcement in flexural member (kg/cm^2) (19)
${}_{W}\tau_A$: allowable shear stress of concrete determined by frame strength surrounding structural wall (kg/cm^2) (25)
${}_{W}\tau_U$: ultimate shear stress of concrete determined by frame strength surrounding structural wall (kg/cm^2) (37)
${}_{m}\psi$: total perimeter of longitudinal tension reinforcement (cm) (19)
π : circumference ratio of circle (17, D, E)

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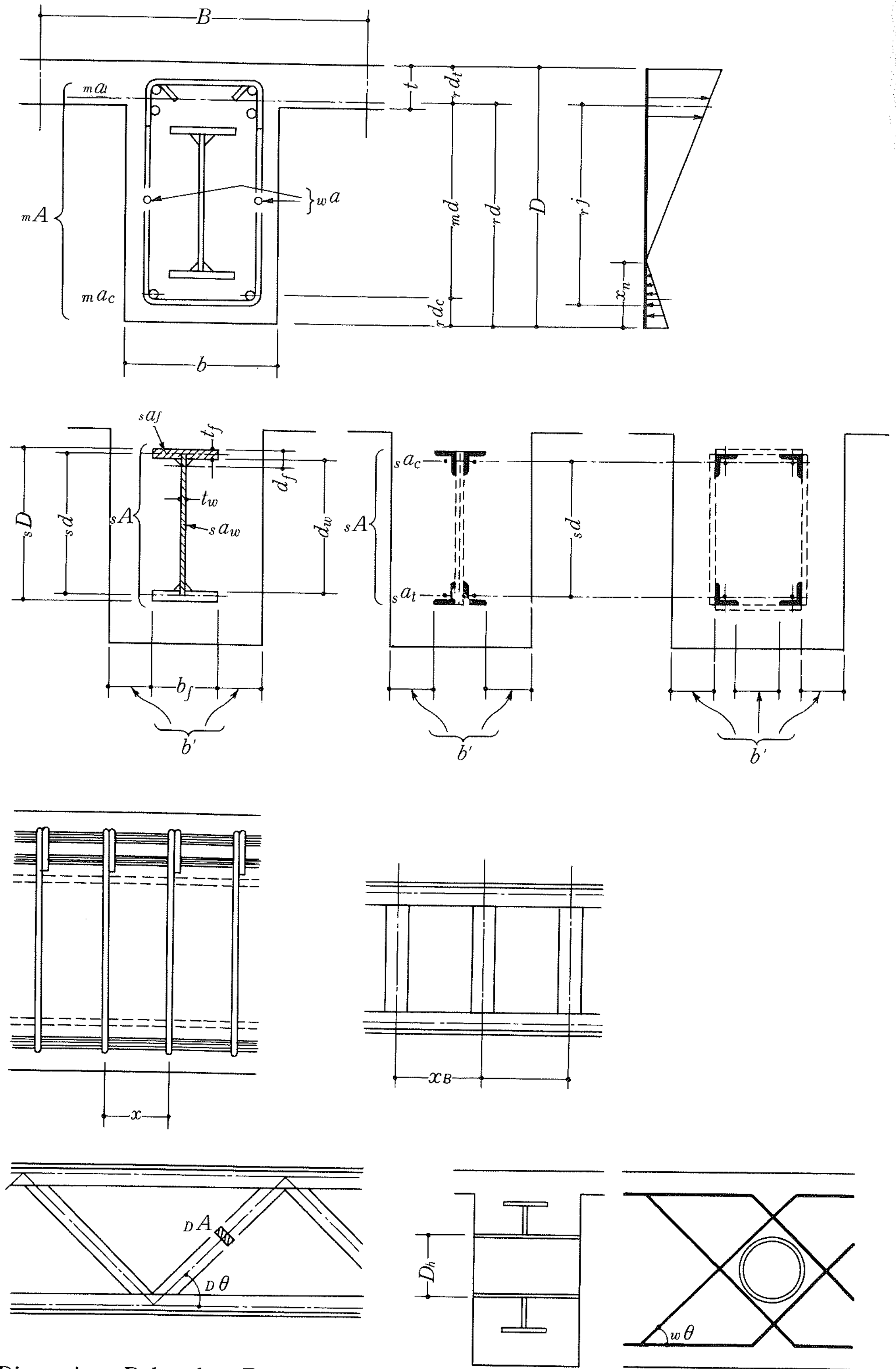


Fig. F1 Dimensions Related to Beams

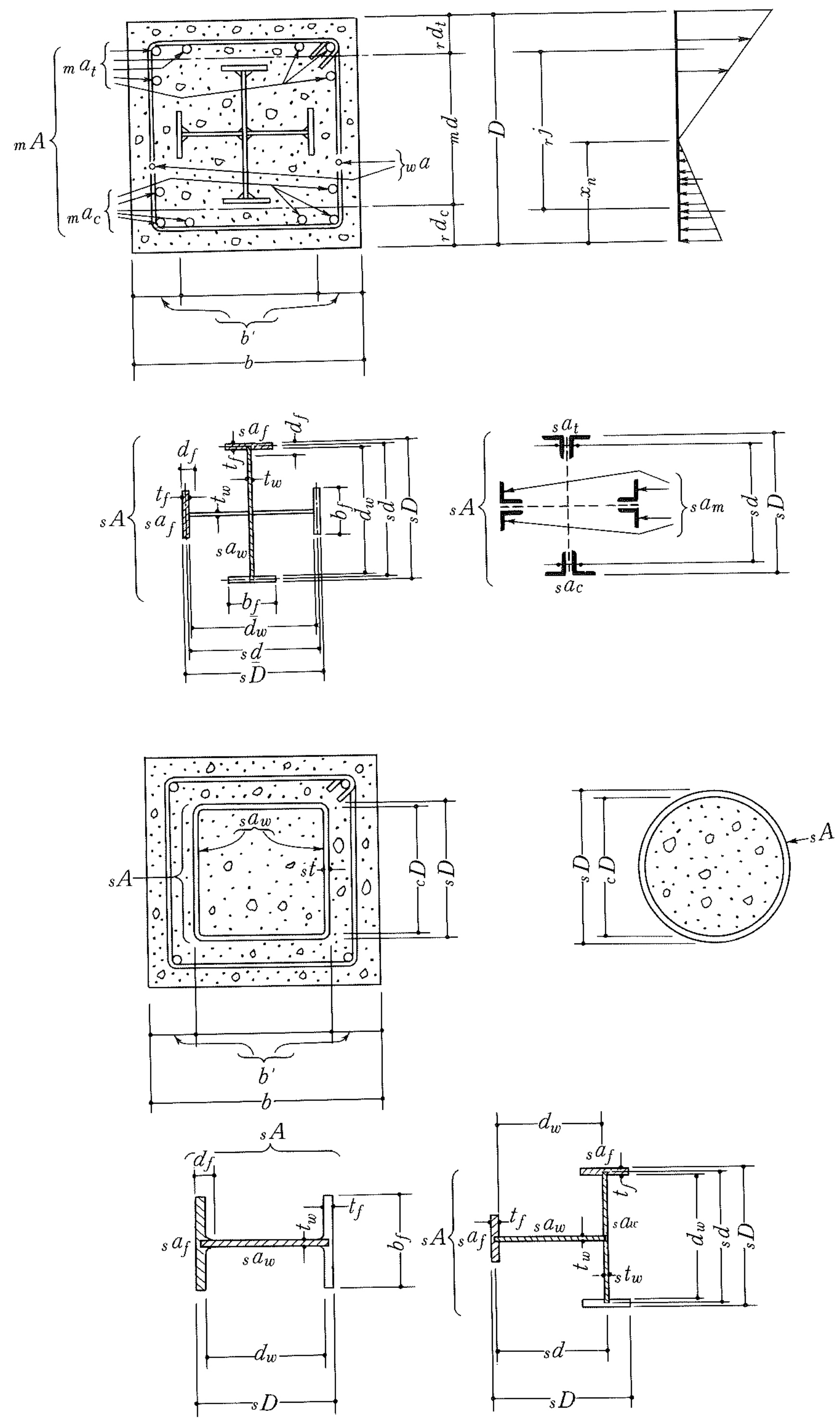


Fig. F2 Dimensions Related to Columns

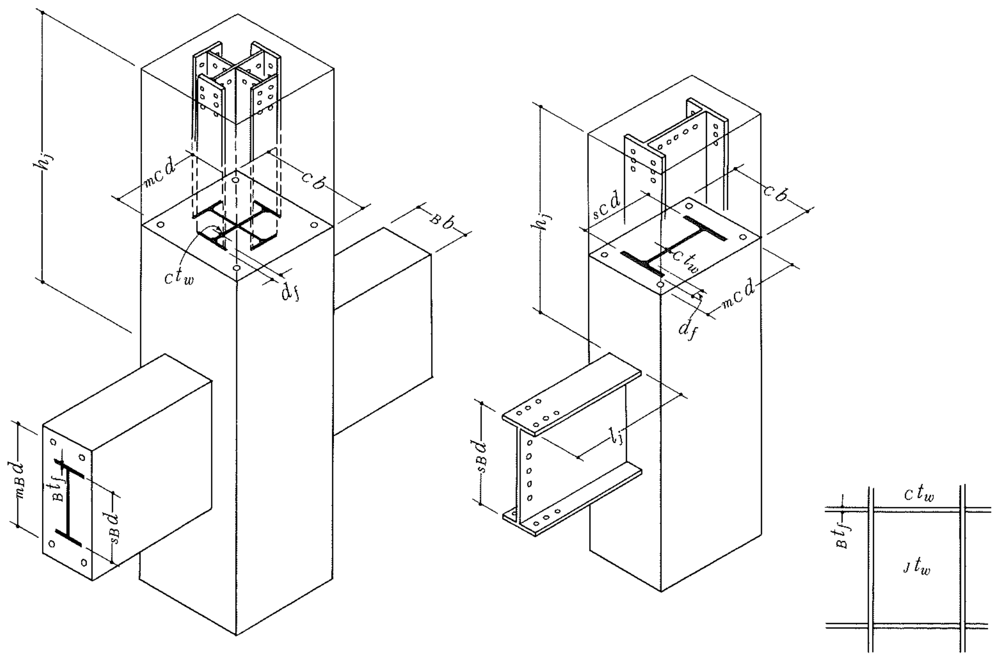


Fig. F3 Dimensions Related to Beam-to-column Connections and Joints

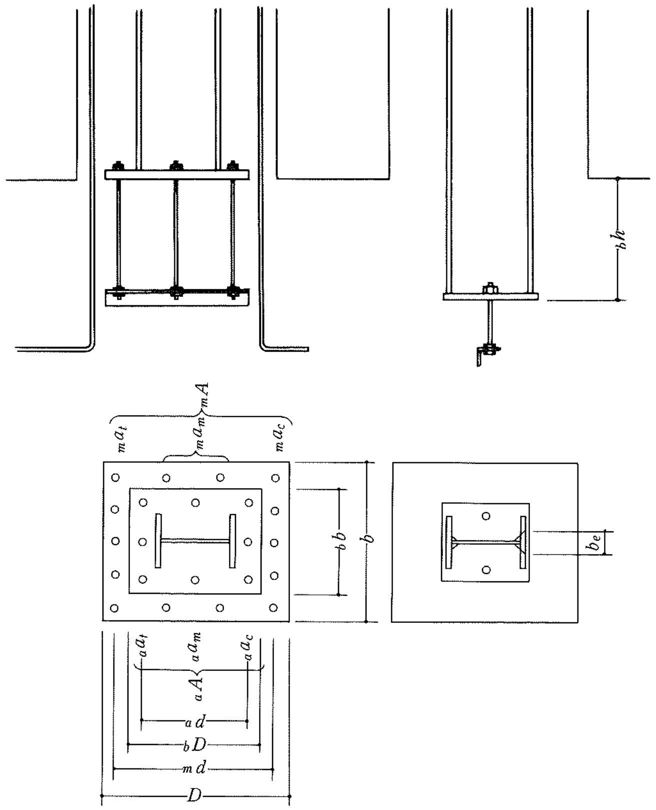


Fig. F4 Dimensions Related to Column Bases

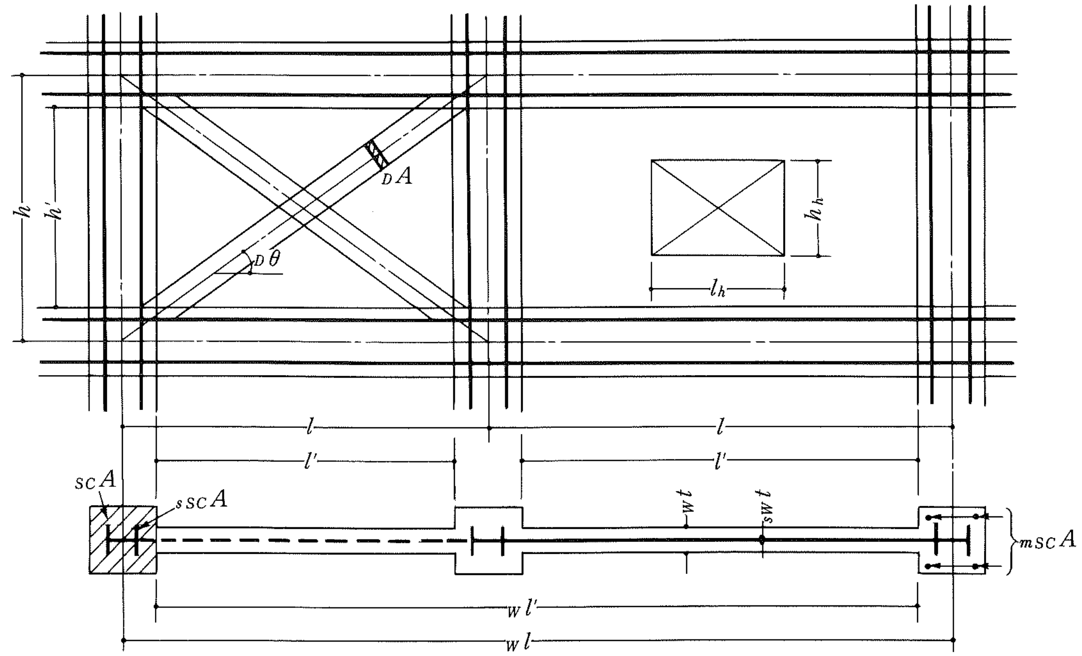


Fig. F5 Dimensions Related to Structural Walls

Brief Commentary
on
AIJ Standards for Structural Calculation of
Steel Reinforced Concrete Structures

Chapter 1 General

Article 1 Scope

A steel reinforced concrete member (abbreviated as SRC member hereafter) is a monolithic concrete encased steel member with longitudinal reinforcing bars, as shown in Figs. 1.1 and 1.2. A structure composed of SRC beams and columns is called an SRC structure. The standards are applied, in general, to the structural design of the SRC structure, and to the SRC portion or the composite steel tube and concrete portion and their connections in mixed structures of steel and reinforced concrete (RC) construction. Structures using special type of shear connectors for steel and concrete, such as composite beams, shall be exempted from the scope.

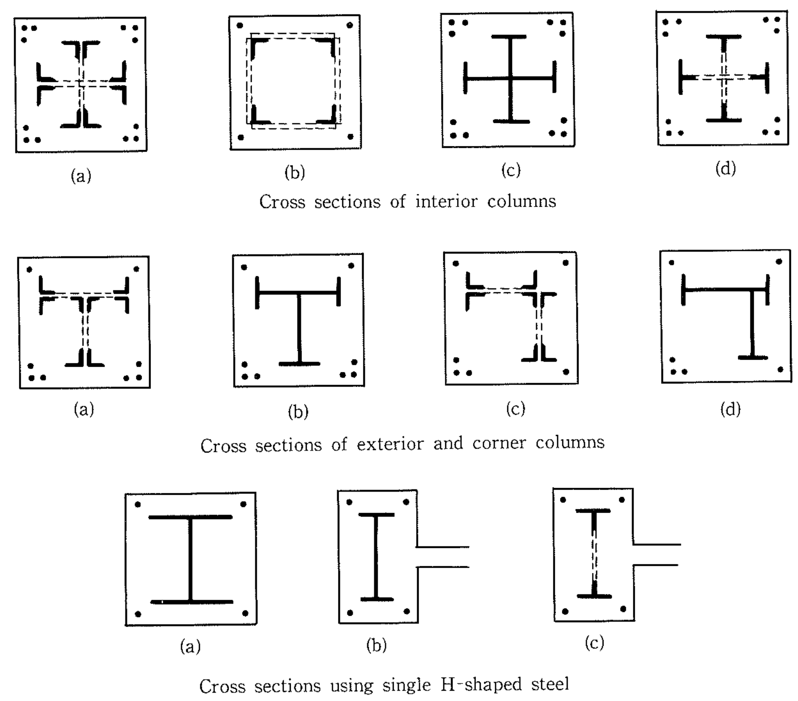


Fig. 1.1 SRC Cross Sections

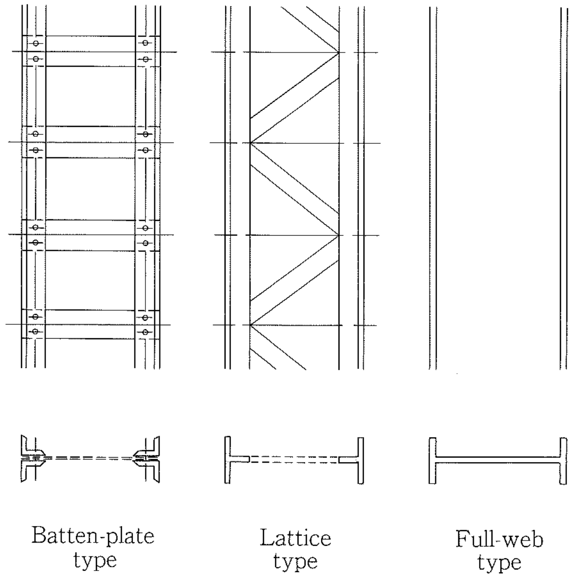


Fig. 1.2 Types of Steel Web

Composite steel tube and concrete structures are classified to filled type, encased type and encased-filled type, as shown in Fig. 1.3, in which both circular and rectangular hollow sections are used.

The Standards are intended for composite steel tube and concrete columns of a framed structure, but may be applied to composite steel tube and concrete beams, braces, and compression members in truss structures.

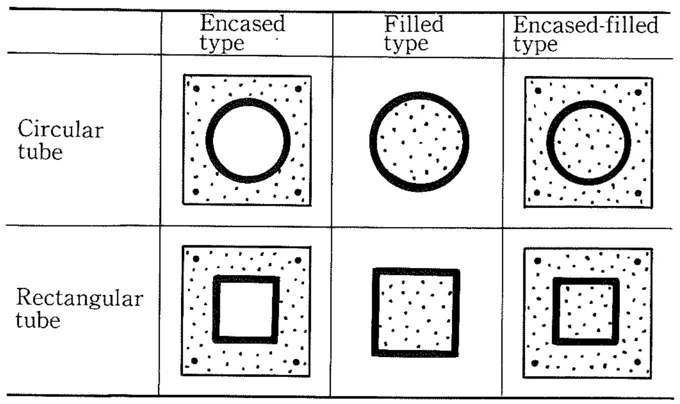


Fig. 1.3 Composite Steel Tube and Concrete Cross Sections Covered by the Standards

Article 2 Procedure of Structural Calculation

Structural calculation is mainly based on the allowable stress design. In earthquake resistant design, some structures are required to examine the horizontal resistance. Design loads shall conform to the Building Standard Law of Japan and its Enforcement Order.

Article 3 Notation

See Appendix F.

Chapter 2 Materials

Article 4 Quality, Shapes and Dimensions of Steel

The use of high-strength steel is limited to Grade SM520. The use of higher strength steel is now under investigation with experiments, and may be introduced in the Standards in the near future.

Chapter 3 Detailed Structural Requirements

Article 7 Detailed Structural Requirements

3. Limiting values for width-thickness or diameter-thickness ratio of the steel section may be relaxed to 1.5 times the values for the bare steel section as shown in Table 7.1, even in the case of members for which sufficient ductility is required. The relaxation is based on restraining effect of concrete to prevent the inward deformation of the steel elements.

Table 7.1 Limiting Values for Width-thickness and Diameter-thickness Ratios of the Steel Section

material	b/t	d/t (beam)	d/t (column)	${}_sD/{}_st$ (square)	${}_sD/{}_st$ (circular)
SS 400	23	107	96	72	150
SS 490	21	99	88	66	129
SM 490	20	91	81	61	109
SM 520	19	87	78	59	100

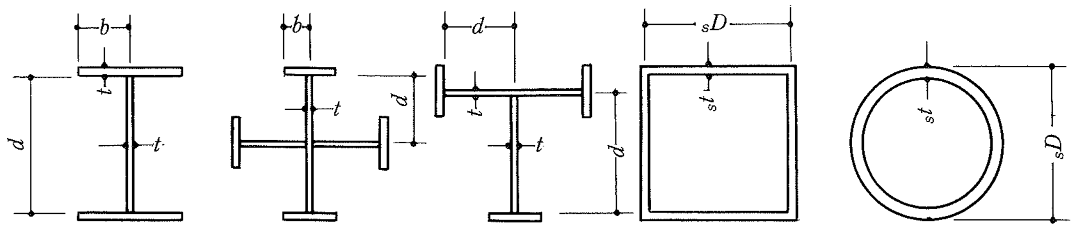


Fig. 7.1 Dimensions of Steel Section

Chapter 4 Allowable Stress Design

Secion 1 General

Article 9 Loads and Their Combination

1. Unit weight of an SRC member per m^3 is taken 100 kg more than that of an RC member.

Article 14 Allowable Stresses

1. Allowable stresses for steel are specified in conformance with Design Standards for Steel Structures (abbreviated as Steel Standards hereafter). The buckling of a member may not be considered, in general, for an SRC member, but the width-thickness ratio of steel sections is limited as indicated in Article 7, Item 3.

2. Allowable stresses for concrete and reinforcing bars are specified in conformance with Standards for Structural Calculation of Reinforced Concrete Structures (abbreviated as RC Standards hereafter).

4. In the shear design of members, the development of bond stresses between steel and concrete is not expected, in general, but the allowable bond stresses are specified for the portion where the examination of bond stress transfer is required. The surface on which gaps may develop between the steel and concrete, such as the lower surface of the steel shape, shall be ignored in the calculation of the bond strength. Allowable bond stress for the steel plate is specified to be one-third the value for plain bars, referring to the test results [14.1] in which the bond strength of the steel plate placed vertically at concrete casting was observed to be 0.45 times the value for plain bars.

Section 2 Calculations for Structural Members

Article 15 General Assumptions

1. Method of superposed strength has been employed since the first edition of SRC Standards published in 1958; the strength of an SRC member was calculated as the sum of strengths of the steel and RC portions. This method is used for beams, columns, beam-to-column connections, joints, column bases and structural walls.

The reliability of the method of superposed strength was proved by the lower bound theorem of the theory of plasticity [15.1, 15.2], and was investigated through experiments [14.1] and analysis [15.3]. The method is simple and easy to use, and the steel and RC portions can be designed independently in accordance with corresponding material standards. When the requirement for the width-thickness ratio of the steel is satisfied according to Article 7, Item 3, local buckling may not be considered.

2. For an SRC member with a batten-plate type open-web steel, the strength of steel portion alone being small, the section may be designed as an RC member counting on the bond stress transfer between steel and concrete. However, such design may be limited to members, such as sub-beams, subjected to long-term load alone, since the bond tends to deteriorate under the repeated loading with a

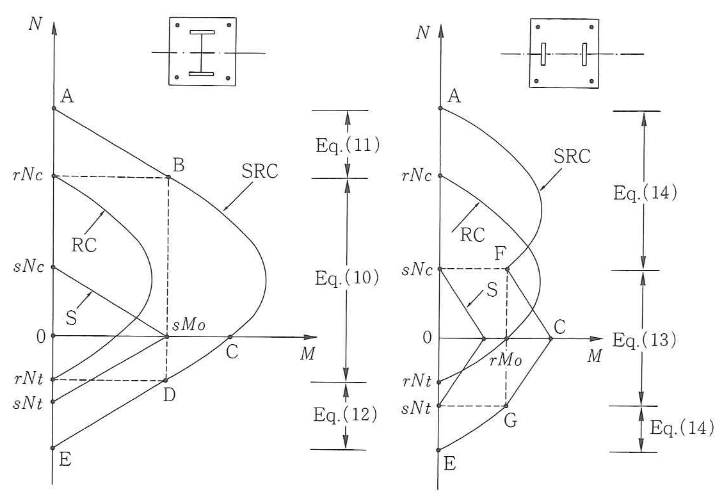
large deformation amplitude.

Article 16 Calculation for Flexural Strength of Beams

The allowable flexural strength given by Eq. (2) is obtained by superposing the allowable bending moments of the steel portion and the RC portion, both carrying no axial force. A larger allowable flexural strength can be obtained by letting the RC portion resist axial compression and flexure, and the steel portion resist axial tension and flexure. Item 2 allows such redistribution of axial forces among constituent elements.

Article 17 Calculation for Axial and Flexural Strengths of Columns

Conventional equations of simple superposition, Eqs. (10) through (12), are derived by moving M-N interaction curve for the RC portion along M-axis by a distance $_sM_o$ and that for the steel portion along N-axis by $_rN_c$; an example for an H-shaped section subjected to strong-axis bending is shown in Fig. 17.1 (a). The Standards recommend a new set of equations for weak-axis bending as shown in Fig. 17.1 (b); Eqs. (13) through (15) are derived by moving the interaction curves for the RC and the steel portions along N- and M-axes, respectively. In addition, Eq. (30) for generalized superposed strength is also usable, which gives the strength interaction curve AHIJKE as shown in Fig. 17.2, larger than either one of the curves ABCDE or AFCGE.



(a) Strong-axis bending

(b) Weak-axis bending

Fig. 17.1 Simple Superposed Strength

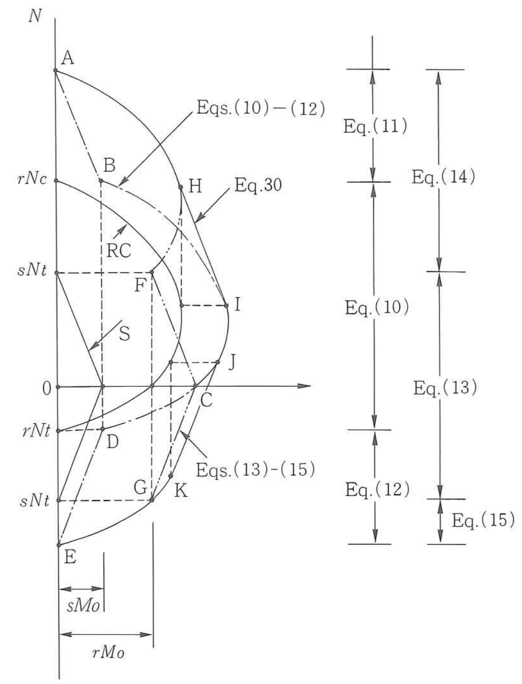


Fig. 17.2 Generalized and Simple Superposed Strengths

1. (5) Allowable stress of concrete for the calculation of the RC portion is reduced by Eq. (29) in proportion to the ratio of the area of steel flange in compression to the gross area of concrete. The reduction considers the following uncertainties: 1) imperfection in concrete-filling due to the existence of the steel shape, 2) error caused by the double calculation of steel stress and concrete stress within the area of the steel shape, 3) error involved in the method of superposed strength caused by insufficient ductility of concrete, and 4) brittle failure of a column with small eccentricity of the axial load. Unsafe error involved in the method of superposed strength is sufficiently compensated by Eq. (29) [15.3].

2. Generalized equation of superposed strength, Eq. (30), gives the largest strength of the cross section. The method of strength calculation was described in the commentary of the old standards, and is formally introduced in the present Standards.

3. The calculation for a column with unsymmetrical bar arrangement may be referred to Ref. [17.1].

4. The calculation for a column subjected to biaxial bending may be referred to Ref. [17.1].

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5. The strength of an SRC long column is calculated by Eqs. (32) through (35), which are modified from Eqs. (10), (11), (13) and (14), based on the study in Refs. [17.2] and [17.3]. Effective buckling length shall be determined in the same way as for the steel column.

6. When a sufficient deformation capacity is required for a column, the axial force should be limited to a value given by Eq. (39). The deformation capacity of 0.01 radian for the chord rotation angle can be assured [17.4].

Article 18 Calculation for Shear and Torsional Strengths of Members

1. In shear design, the bond between steel shape and concrete is neglected. Formulas were derived including the contribution of the steel to increase the cracking strength in the member containing full-web type or truss type steel. The ultimate shear strength can be evaluated by the method of superposition, on the basis of the experiments conducted by AIJ Committee for SRC Structures [17.4]. That is,

$$Q_U = {}_sQ_U + {}_rQ_U \qquad \text{(C 18.1)}$$

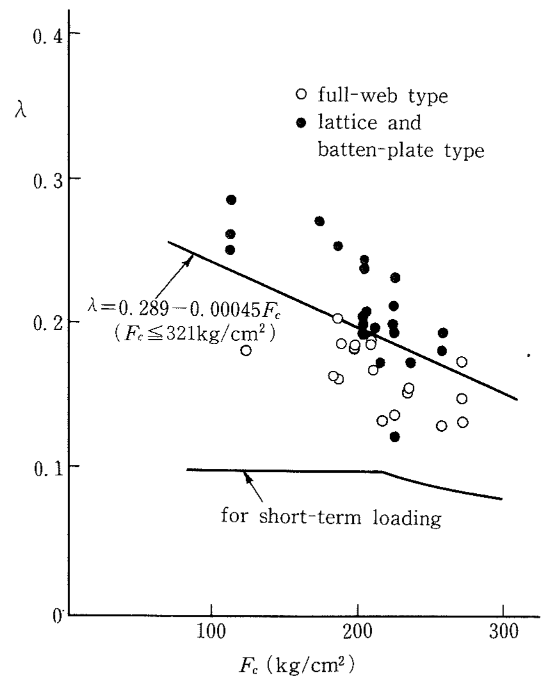

Fig. 18.1 Relation between Shear Bond Failure Strength and Concrete Compressive Strength

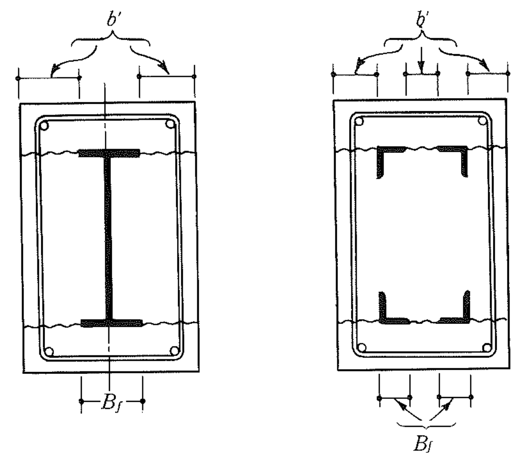

Fig. 18.2 Effective Concrete Width b' at Steel Flange of Beam

where ${}_sQ_U$ = strength of the steel portion determined as the smaller of the shear strength carried by the web and the shear at ultimate flexural strength of the steel portion, and ${}_rQ_U$ = strength of the RC portion determined as the smaller of the ultimate shear strengths corresponding to shear failure and shear bond failure. The shear bond failure strength is given by

$${}_rQ_U = b \cdot {}_rj\{(b'/b) \cdot \lambda \cdot F_c + \mu \cdot {}_wp \cdot {}_w\sigma_Y\} \qquad \text{(C 18.2)}$$

where μ = 1 and λ shown in Fig. 18.1 are obtained from the tests. The strength corresponding to shear failure is given in RC Standards.

2. (1) The shear design for beams under the long-term loading condition should be carried out so that the allowable shear strengths of the steel and the RC portions must be greater than corresponding design shear forces.

(3) The first expression in Eq. (44) gives the allowable shear strength specified in RC Standards. However, the term $0.5{}_wf_t({}_wp-0.002)$ representing the effect of stirrups in an RC member is replaced by $0.5{}_wf_t \cdot {}_wp$ based on the test results of SRC beams. The second expression of Eq. (44) for the allowable strength at shear bond failure is obtained from Eq. (C 18.2) by substituting the allowable shear stresses of concrete and web reinforcement under the short-term loading condition, $f_s = F_c/20$ and ${}_wf_t = {}_w\sigma_Y$, respectively, with $\lambda = 0.1$ and $\mu = 1$, which gives the lower bound to the strengths observed in the tests. Figure 18.2 shows the effective width of concrete b'.

When Eq. (44) is applied for the long-term stress condition, it has the factor of safety equal to 1.5 to the lower bound of the strengths observed in the tests, since the values of f_s and ${}_wf_t$ are both 1/1.5 times the corresponding values for the short-term loading condition.

(4) For building structures in which horizontal load resistance shall be checked in the second stage earthquake resistant design, the design shear force due to the earthquake load may be distributed to the steel and RC portions according to Eqs. (46) and (47) in the first stage design. However, if the lateral load resisting capacity is not examined, the RC portion should be designed against the shear force determined by Eq. (48), in order to secure sufficient ductility by making the flexural failure precede the shear failure. When the shear in Eq. (48) becomes too large, Eq. (49) may be used instead. The steel portion has large ductility even if it fails in shear, and thus the design shear force can be determined by Eq. (46).

3. (1) The shear design of columns under the long-term load should be carried out so that no shear cracks occur. For the short-term load, the design procedure is the same for columns and beams.
(2) Contribution of concrete to the lower bound of the shear-cracking load of an RC column given in RC Standards is as follows:

$$_{r}Q_{AL}=b\cdot{}_{r}j\cdot\alpha\cdot f_s \tag{C 18.3}$$

Experimental results show that the lower bound of the cracking load by the shear bond failure is obtained from Eq. (C 18.2) by neglecting the term of stirrups. Substituting $F_c = 30f_s$ (f_s: allowable shear strength of concrete under the long-term loading condition) and $\lambda = 0.1$ into Eq. (C 18.2) with neglecting the term of stirrups leads to

$$_{r}Q_{AL}=3b'\cdot{}_{r}j\cdot f_s \tag{C 18.4}$$

and thus Eqs. (58) and (59) are derived.

In the full-web type encased steel, the web carries some shear before cracking, and thus the shear-cracking load increases. The effect is expressed by β in Eq. (57). The first expression of Eq. (60) for the full-web type steel is derived by equating the shear strains in the concrete and the web at the cracking load using some approximations; i. e., the shear stress is assumed uniformly distributed in the web in the first expression of Eq. (60), and the maximum shear stress in the flange is assumed 1.5 times the average value in the derivation of the second expression of Eq. (60). The expressions of β for other type of webs can be obtained in a similar manner.
(4) Figure 18.3 shows the effective concrete width in Eq. (63).
(5) The same shear design concept is used in beams and columns. Normally, intermediate loads do not exist in columns, and Eq. (64) is thus applied. Equation (48) must be applied to columns with intermediate loads, such as basement columns subjected to earth pressure. If the value of $_{r}Q_d$ given by Eq. (64) or (48)

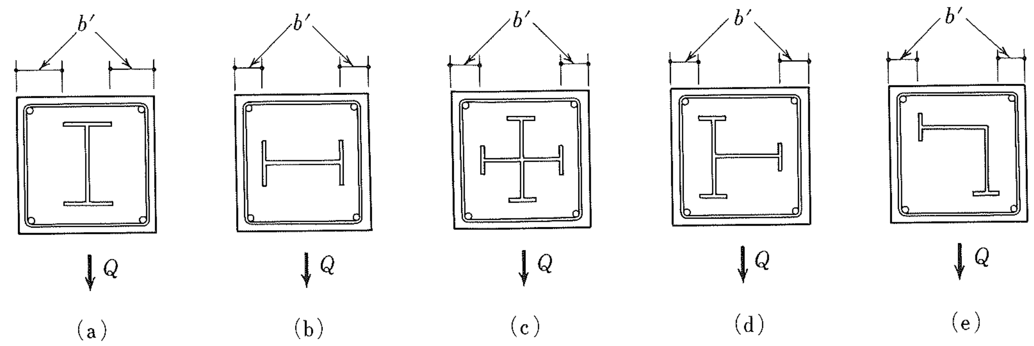

Fig. 18.3 Effective Concrete Width b' at Steel Flange of Column

is too large, Eq. (49) may be applied instead. The axial forces listed in Table 14 may be used in the calculation of ultimate flexural strengths of the RC portion. The axial forces under the short-term loading condition and at the ultimate state are different in general, but they are assumed the same for simplicity. If adjacent beams yield prior to column yielding, the shear force in the column may be calculated from the column end moments at the beam yield moments. However, since this is an approximation, this method applies to only one end of the column, even if all adjacent beams yield at both ends of the column.

The ultimate flexural strength of the RC portion for the calculation of the design shear force is calculated by Eqs. (111) through (113), in which $_{c}\gamma_U$ shall be taken equal to unity in order to obtain the conservative design shear force.

4. (2) The allowable shear strength of the steel portion of a beam with an opening $_{s}Q_A$ is calculated for a uniform or parabolic stress distribution in the web, with or without flange around the opening to strengthen the web, respectively.
(3) The first expression in Eq. (70) for the allowable shear strength of the RC portion of a beam with an opening includes the effect of the opening by a factor $(1-1.6D_h/D)$ in the first term. In addition, the shear bond failure must be checked by the second expression of Eq. (70).

Article 19 Bond for Reinforcing Bars and Steel

1. (1) The shear design for SRC members in the Standards is made without expecting the bond between the shaped steel and the concrete, and thus it is not needed to check the bond for the steel portion. The bond for the reinforcing bars must be investigated according to RC Standards.

Article 20 Beam-to-Column Connections

2. The shear force ${}_{P}Q_{c}$ acting on the panel of a beam-to-column connection shown in Fig. 20.1, subjected to equal moments ${}_{B}M$ at both adjacent beam ends, is approximately given by

$$ {}_{P}Q_{c}=2({}_{B}M/{}_{mB}d)\cdot(h'/h) \quad \text{(C 20.1)}$$

If the moments at the right and left beam ends are ${}_{B}M_{1}$ and ${}_{B}M_{2}$, the panel shear is given by

$$ {}_{P}Q_{c}=\{({}_{B}M_{1}+{}_{B}M_{2})/{}_{mB}d\}\cdot(h'/h) \quad \text{(C 20.2)}$$

On the other hand, the test results show that the ultimate shear strength of a panel is given by a superposed strength type formula:

$$ {}_{P}Q_{c}={}_{c}\tau_{U}\cdot{}_{c}A_{e}+({}_{s}\sigma_{Y}/\sqrt{3})\cdot{}_{s}A \quad \text{(C 20.3)}$$

where ${}_{c}A_{e}$ and ${}_{s}A$ = horizontal areas of the concrete panel and the steel panel, respectively, ${}_{s}\sigma_{Y}$ = yield stress of the steel panel, and ${}_{c}\tau_{U}$ = nominal shear strength of the concrete panel which is assumed to be uniformly distributed in the concrete area ${}_{c}A_{e}$. In Fig. 20.2, the values of ${}_{c}\tau_{U}$ obtained from Eq. (C 20.3) by substituting the test results into ${}_{P}Q_{c}$ are plotted against F_{c}, and it is observed that the relation ${}_{c}\tau_{U}=0.3F_{c}$ gives the lower bound to the test results [20.1]. Thus, the condition that Eq. (C 20.3) is greater than Eq. (C 20.2), with ${}_{c}V_{e}={}_{c}A_{e}\cdot{}_{mB}d$, ${}_{s}V={}_{s}A\cdot{}_{sB}d$ and ${}_{mB}d\fallingdotseq{}_{sB}d$,

$$0.3F_{c}\cdot{}_{c}V_{e}+({}_{s}\sigma_{Y}/\sqrt{3})\cdot{}_{s}V\geqq({}_{B}M_{1}+{}_{B}M_{2})\cdot h'/h \quad \text{(C 20.4)}$$

gives the lower bound to the shear strength of the panel.

(1) The panel of a beam-to-column connection under the long-term loading condition should be designed to prevent diagonal cracking. As in the case of shear design of columns, it is considered that the shear cracking strength becomes $(1+\beta)$ times the shear cracking strength of the concrete panel alone taking the contribution of the steel panel into account, and then the following equation is derived:

$$ {}_{c}\tau_{cr}\cdot{}_{c}A\cdot{}_{mB}d(1+\beta)=({}_{B}M_{1}+{}_{B}M_{2})\cdot h'/h \quad \text{(C 20.5)}$$

where ${}_{c}\tau_{cr}$ = shear cracking strength of concrete. Substitution of ${}_{c}\tau_{cr}=F_{c}/10$ into Eq. (C 20.5) with $F_{c}=30f_{s}$ and ${}_{c}V={}_{c}A\cdot{}_{mB}d$ leads to Eq. (75), where ${}_{c}V$ denotes the volume of concrete resisting the shear, which is calculated by taking

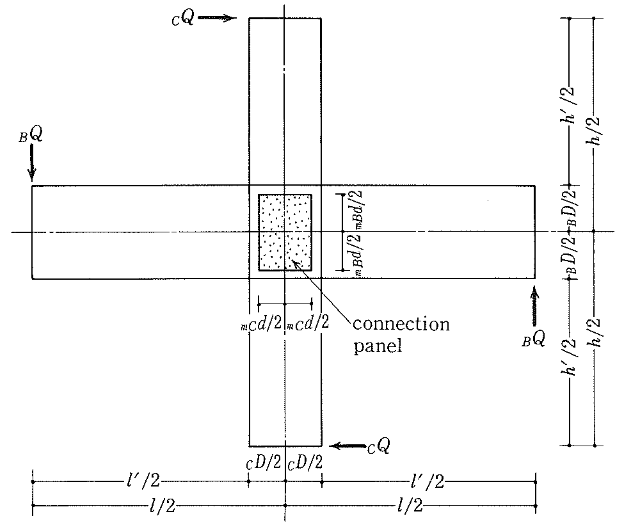

Fig. 20.1 Cruciform Frame under Earthquake Loading

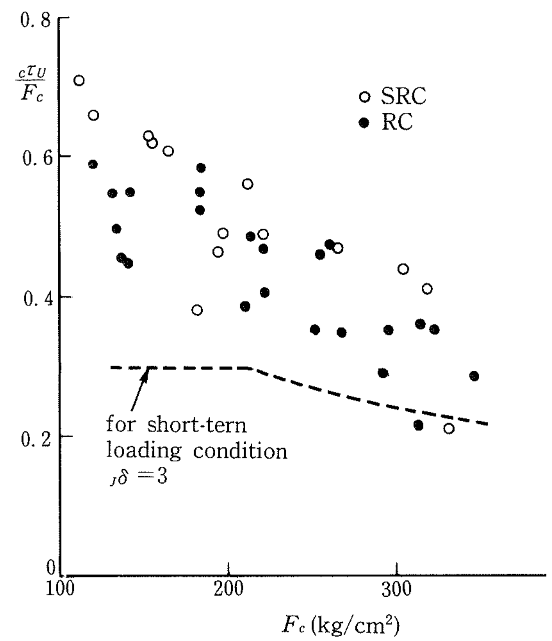

Fig. 20.2 Shear Strength of Concrete Panel in Beam-to-column Connection

the column width for the width of the concrete panel.

(2) The panel under the short-term loading condition should be designed to prevent the shear failure. Substituting $F_c = 20f_s$ and ${}_s\sigma_Y = \sqrt{3}\,{}_sf_s$ into Eq. (C 20.4) and adding ${}_wp \cdot {}_wf_t$ of the contribution from hoops lead to a formula for the cross-shaped connection as follows:

$$(6f_s + {}_wp \cdot {}_wf_t)\,{}_cV_e + {}_sf_s \cdot {}_sV \geqq ({}_BM_1 + {}_BM_2) \cdot h'/h \qquad \text{(C 20.6)}$$

where f_s = allowable shear stress of concrete under the short-term loading condition. For T-shaped or L-shaped connections, the restraining effect given to the panel from surrounding members is less than that of the cross-shaped connection, and the value of ${}_c\tau_U$ becomes 2/3 or 1/3 of the value for the cross-shaped connection. Replacing $6f_s$ by $2f_s \cdot {}_J\delta$ leads to Eq. (76) where the values in Table 16 are used for ${}_J\delta$.

3. If the ratio between the bending moments carried by the steel portion and the RC portion differs substantially at the column end and the beam end surrounding the connection, smooth stress transfer from the beam to the column may not be secured. According to the tests of variable moment ratio, if the moment carried by the column steel is greater than 40% of the total moment acting on the column, the full strength of the column can be developed [20.2]. Thus, the Standards state that the check for the stress transfer is not needed if Eq. (77) is satisfied. In the connection between the RC beam and the SRC column, the same problem occurs in the stress transfer in concrete, and thus Eq. (78) is given.

Article 21 Joints

1. (1) When the lateral load resisting capacity is checked in the second stage earthquake resistant design with consideration of the ductility, the stress carried by the steel joint may be determined by Eq. (80) from the design shear force by the earthquake load. The value of ${}_sM_d/M$ often differs at the location of joints even in the same member, and the value used for the flexural design of the member end nearest to the joint should be taken.

(2) When the lateral load resisting capacity is not checked, the design stresses for the joint should be determined by Eqs. (81) and (82), in order to assure sufficient ductility by preventing the failure of the joint although the member yielding occurs. Equation (81) is derived from the condition that the beam joint should not fail under the stress state shown in Fig. 21.1 (a), which is the sum of the stresses due to intermediate loads and the earthquake load shown in (b) and (c), respectively, where ν_j is the stress amplification factor to compensate the increase in the member strength due to strain hardening and yield stress larger

than the nominal value, and Y denotes the yield ratio of steel material. The values ν_j and Y are given in Table 17. If the design stresses given by Eqs. (81) and (82), derived under the condition that the plastic hinges form at both ends of the member, become unreasonably large, Eq. (83) may be used for simplicity.

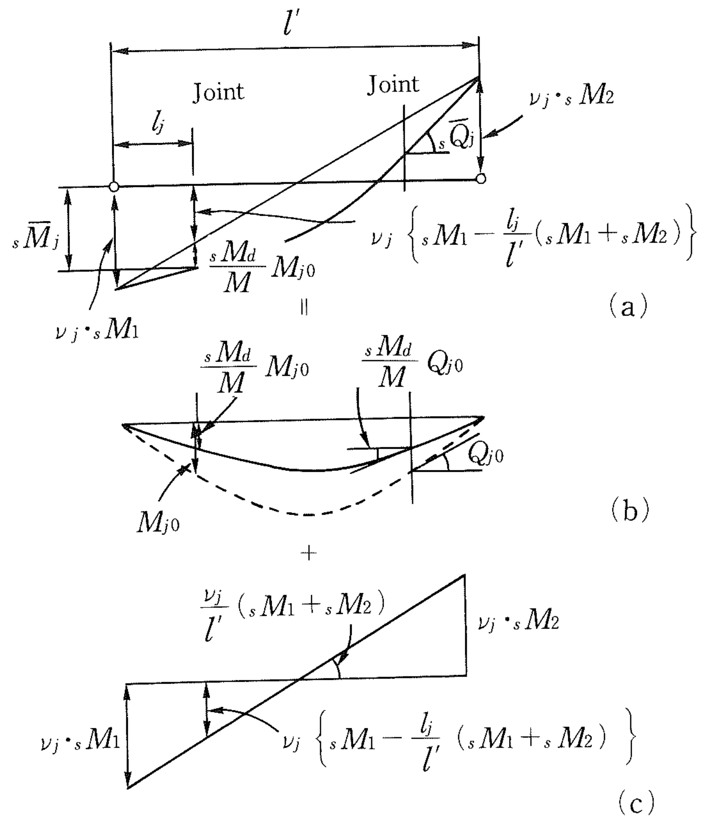

Fig. 21.1 Design Stresses at Steel Beam Joint

Article 22 Column Bases

2. (1) In the case of a bare type column base of which steel portion is set on the surface of the RC foundation beam as shown in Fig. 22.1, the stresses at the base are transferred to the foundation through the steel column base consisting of base plate, anchor bolts and the RC portion surrounding the base plate. The strength of such a column base is calculated by applying the superposed strength method, the same as that of the columns.

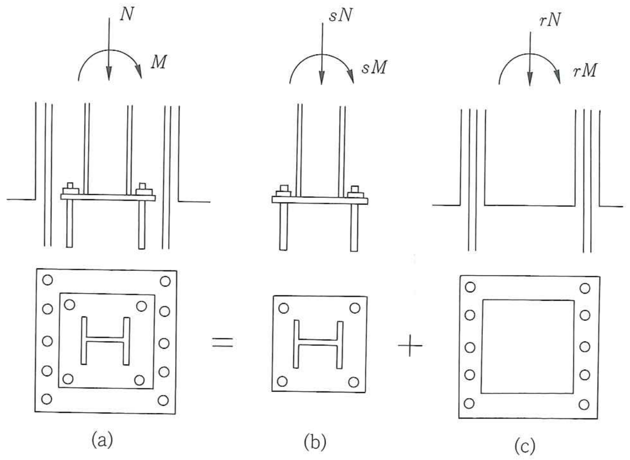

Fig. 22.1 Strength of Bare Type Column Base

(2) The strength of the steel portion of a column base is obtained as the sum of the strengths of anchor bolts and concrete below the base plate as shown in Fig. 22.2, where it is assumed that the anchor bolts resist tension only. The M-N interaction curves for the design formulas are illustrated in Fig. 22.2 (b).

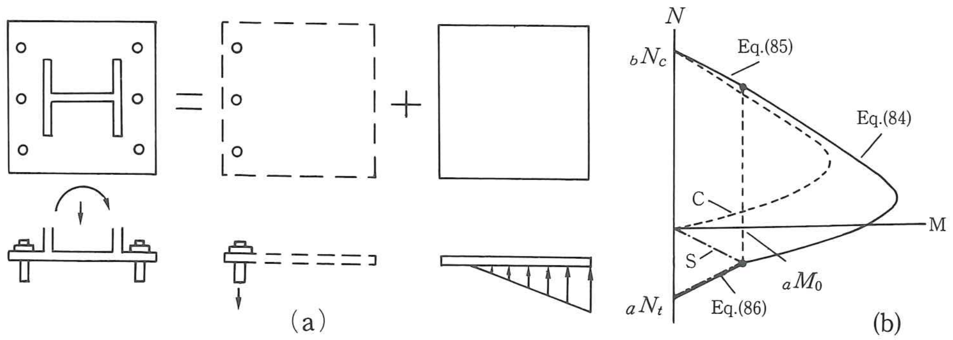

Fig. 22.2 Strength of Steel Portion of Column Base

(3) The strength of the RC portion surrounding the base plate is obtained by the method of superposition as shown in Fig. 22.3, which illustrstes the M-N interaction of the design formulas.

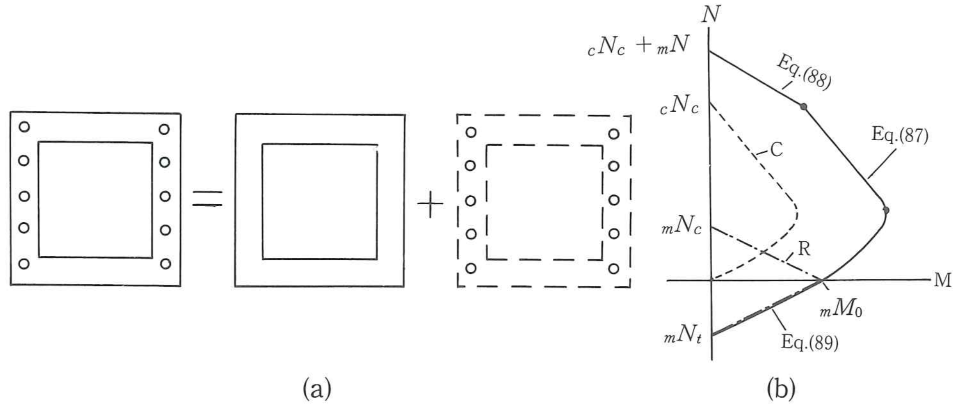

Fig. 22.3 Strength of Covering RC Portion

3. (1) In the column base of which steel portion is embedded into the RC foundation beam, the moment M at the base plate becomes smaller than $_sM$ at the beam surface because the side pressure acts on the steel portion from the concrete of the beam (Fig. 22.4). The side pressure can be separated into two parts, i. e.,

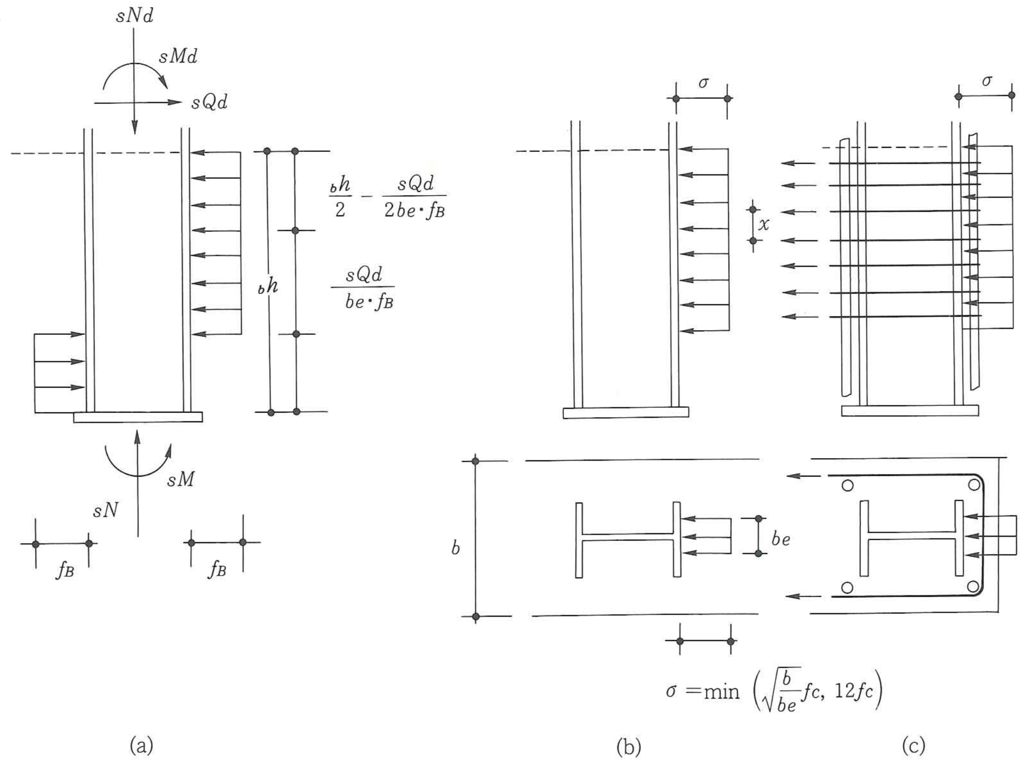

Fig. 22.4 Strength of Embedded Type Column Base

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contribution to the shear resistance $_sQ$ and the flexural resistance, shown in Fig. 22.4 (a). Equation (90) is derived from the equilibrium in the bending moments at the base plate. The effective width b_e is determined from the flexural stiffness of the steel flange, and the allowable bearing stress of concrete f_B is given by Eq. (91). In the case of an exterior column as shown in Fig. 22.4 (c), the bearing stress from the steel portion must be carried by hoops, and thus the last term on the right hand side of Eq. (91) is needed.

Article 25 Structural Walls

2. (1) The allowable shear strength of a structural wall under the short-term loading condition is taken equal to the larger strength of the shear cracking strength $_wQ_{A1}$ (which gives the lower bound to the test results) and the strength developed after the cracks occur (Fig. 25.1 (a)). Until the cracks occur, steel braces made of flat bars or shaped steel and steel plates carry the shear force in proportion to their elastic stiffnesses, and thus the shear cracking strength becomes higher accordingly (b). After the cracks occur, the ultimate strength of the brace or the steel plate will be developed, and the method of superposition can be applied to estimate the strength as indicated by Eq. (94) ((c) and (d)).

(2) The effect of openings in the wall can be evaluated by a reduction factor r (e). The behavior of the wall with rather small opening is similar to that of the wall without opening, and the strength is evaluated by Eq. (95). On the other hand, if the opening becomes large, the behavior becomes similar to that of a frame consisting of columns with side walls and beams with sagging walls, to which Eq. (97) applies (f). The strength of a frame with a structural wall is considered as the sum of the strength of the wall $_wQ_o = r\ \min\{f_s', ({}_wp\cdot{}_wf_t + {}_w\tau_A)\}{}_wt\cdot l'$ (g) and the strength determined from the shear strengths of beams and columns, $R_A = \min(\Sigma_{cs}Q_A,\ \Sigma_{BS}Q_A\cdot l/h)/2$ (h). However, beams and columns not surrounded by the walls are considered later in connection with Eq. (100)

The strength associated with the expansion of the wall is considered as the sum of the shear carried by a truss consisting of wall reinforcements and a diagonal strut of concrete, $r\cdot{}_wp\cdot{}_wf_t\cdot{}_wt\cdot{}_wl'$ (j) and the shear resistance provided by the surrounding frame restraining the expansion of the concrete strut, $r\cdot{}_w\tau_A\cdot{}_wt\cdot{}_wl'$ (k).

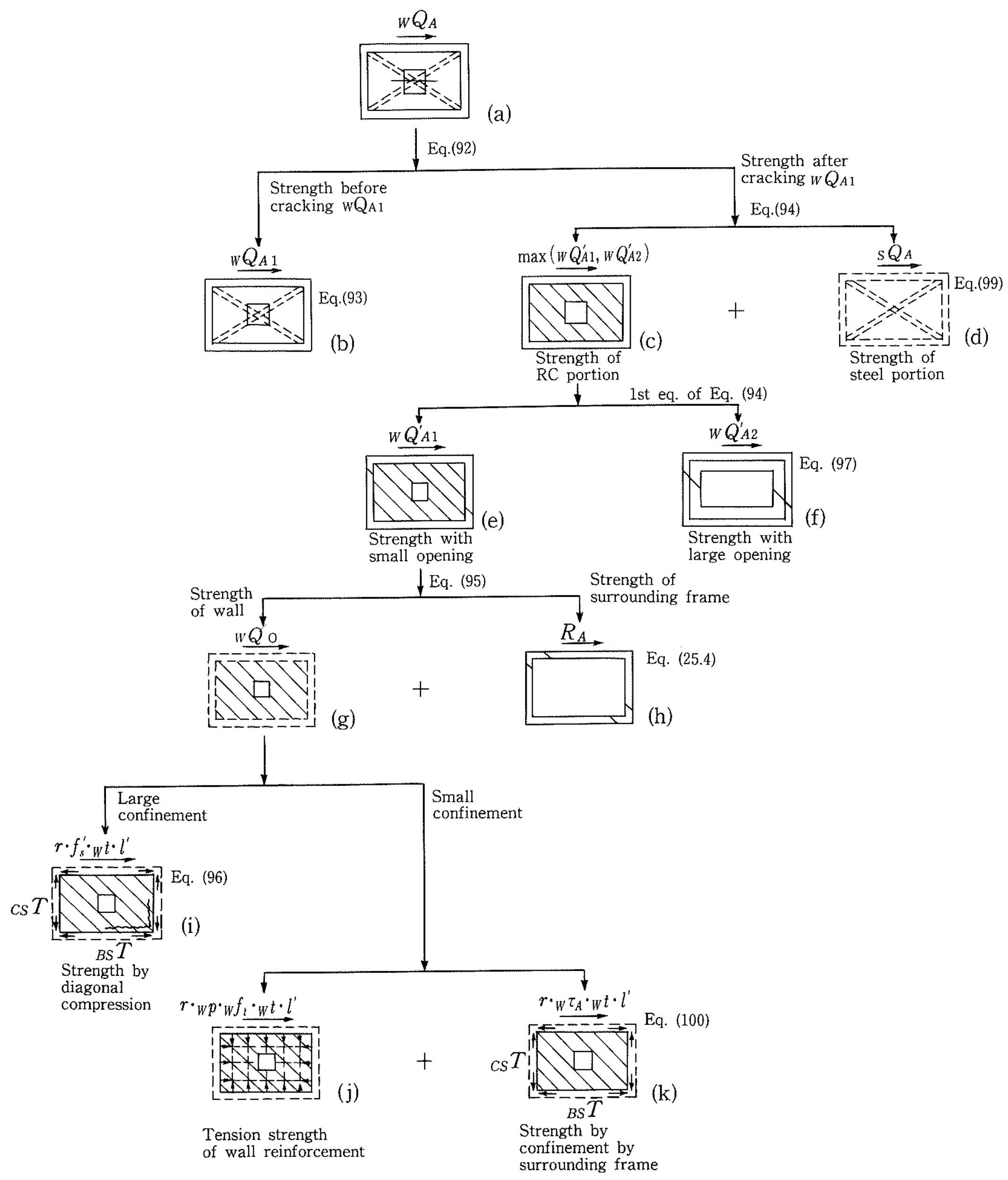

Fig. 25.1 Formatinn of Design Formulas for Structural Walls

(4) The shear strength of the wall determined from the compressive strength of the concrete diagonal strut is given by $f_s' = \min(0.3f_c,\ 3.51\sqrt{f_c})$. The values for f_s' are derived as follows: The equilibrium between the shear stress τ_c and the normal stress σ_c acting on the diagonal strut with inclination angle of 45° becomes $\tau_c = \sigma_c/2$. It is assumed that the compressive strength of the wall after large diagonal cracks is taken equal to $F_c/2$ due to the strength deterioration, and then the ultimate value for τ_c becomes $F_c/4$. It is further assumed that the allowable strength for the short-term loading condition may be 80 % of the ultimate value. Thus, in view of the allowable compressive stress of the short-term loading condition $f_c = 2F_c/3$, the shear strength becomes $0.8\times(F_c/2)/2 = 0.8\times(3f_c/2)/4 = 0.3f_c$. On the other hand, the average value of the test data by Tomii and Esaki [25.1] for the ultimate shear strength of RC walls which show slip type failure is $3.6\sqrt{F_c}$, and thus $0.8\times3.6\sqrt{F_c} = 0.8\times3.6\sqrt{3f_c/2} = 3.51\sqrt{f_c}$ (Fig. 25.2).

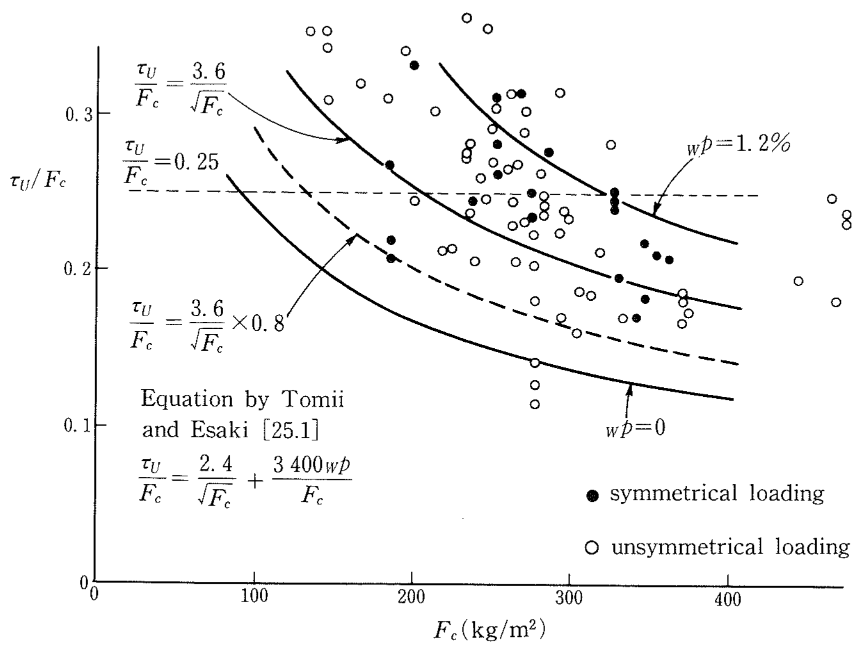

Fig. 25.2 Slip Failure Strength of Reinforced Concrete Walls

The strength of a surrounding frame given in Eq. (100) is associated with the flexural and shear yielding of beams and columns. The values of factors ${}_{BS}\beta_b$, ${}_{CS}\beta_b$, ${}_{BS}\beta_s$ and ${}_{CS}\beta_s$ are related to the stress conditions of beams and columns subjected to the internal pressure from the wall. The real conditions are rather

complicated, but the values in Eq. (25.1) may be used, which are derived for simply-supported members subjected to uniformly distributed internal pressure.

$$\begin{aligned} &{}_{BS}\beta_b=16, \quad {}_{CS}\beta_b=16 \\ &{}_{BS}\beta_s=2, \quad {}_{CS}\beta_s=2 \end{aligned} \qquad \text{(C 25.1)}$$

(5) The strength of a beam or a column with the wall is determined as the smaller of flexural and shear strength (Eqs. (101) and (102)).

(6) The values of the reduction factor r are obtained from the tests of walls with a small opening. The factors r_1 and r_2 are applied to the walls with an approximately square opening and long horizontal opening, respectively, which have been conventionally used and shown in RC Standards. The factor r_3 is for the walls with a long vertical opening.

3. The tension force equal to the horizontal or the vertical component of the compression force carried by the concrete diagonal strut minus the force in the wall reinforcement acts on the steel beam or the steel column, respectively (Fig. 25.1 (k)).

Chapter 5 Calculation of Lateral Load Resisting Capacity

Section 3 Calculation of Ultimate Strength of Members and Connections

Article 30 General Assumptions

Calculation of the ultimate strength of members and connections is based on the superposed strength method. Various factors appearing in the strength formulas are determined to give average values of the ultimate strength obtained from the tests.

Article 32 Ultimate Flexural Strength of Members

(1) Equations (108) through (110) correspond to Eqs. (10) through (12) in Article 17.

(2) The ultimate flexural strength of the RC portion is obtained by superposing the strength of reinforcing bars on the strength of the concrete portion (Fig. 32.1).

(3) The value of ${}_cM_U$ is given in Table Bl in Appendix, in view of Fig. 32.2. The factor ${}_cr_U$ is the same as f_c'/f_c in Article 17, and Eq. (114) is an empirical formula

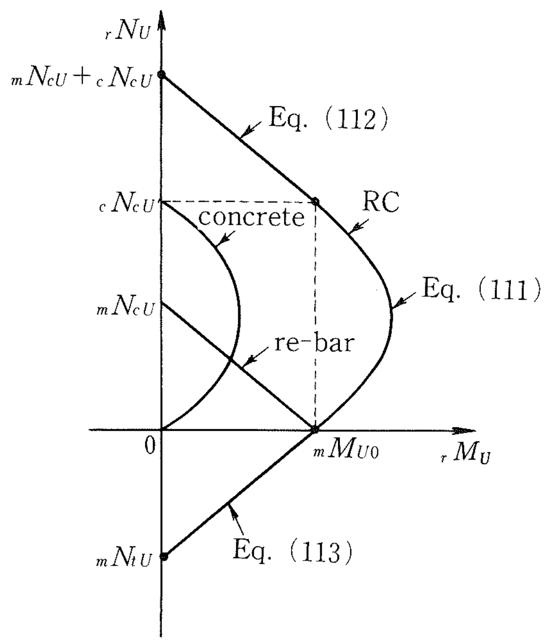

Fig. 32.1 Ultimate Flexural Strength of RC Portion

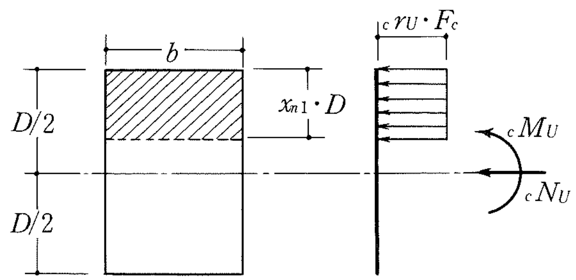

Fig. 32.2 Stress Distribution in Concrete at Ultimate State

derived from the tests for the ultimate strength of members.

(4) The value of ${}_rM_U$ is given in Table B 2 in Appendix, in view of Figs. 32.3 and 32.4.

(5) The value of ${}_sM_U$ is given in Table B3 in Appendix, as schematically described in Figs. 32.5 through 32.9.

3. Equation (115) is based on the generalized superposition of the strengths of concrete, longitudinal reinforcement and steel, and Eq. (116) on the supperposition of the strengths of the steel and RC portions.

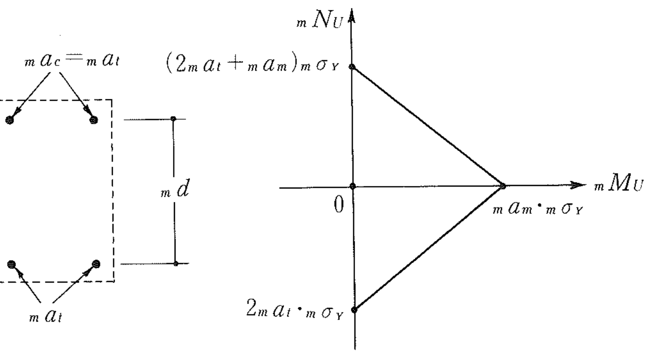

Fig. 32.3 Ultimate Flexural Strength by Longitudinal Reinforcement

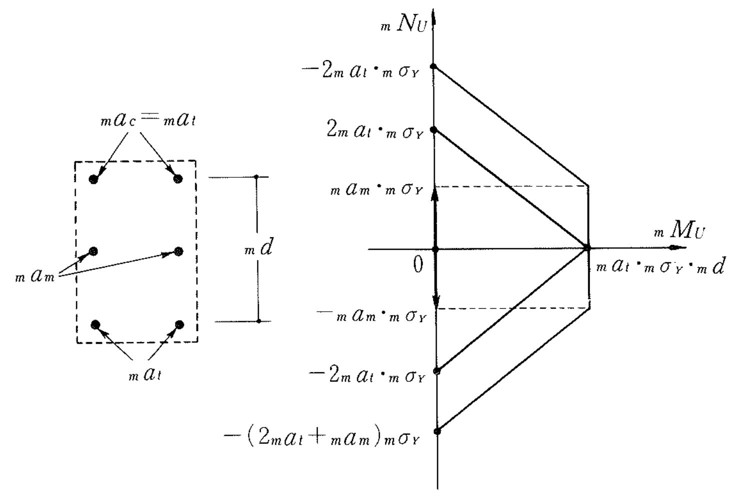

Fig. 32.4 Ultimate Flexural Strength by Longitudinal Reinforcement with Intermediate Bars

6. Equation (118) expresses the equation of generalized superposition for the steel portion subjected to the tension force. The equation should be applied only to a member so detailed that the tension force can be equilibrated with the compression force in the concrete portion.

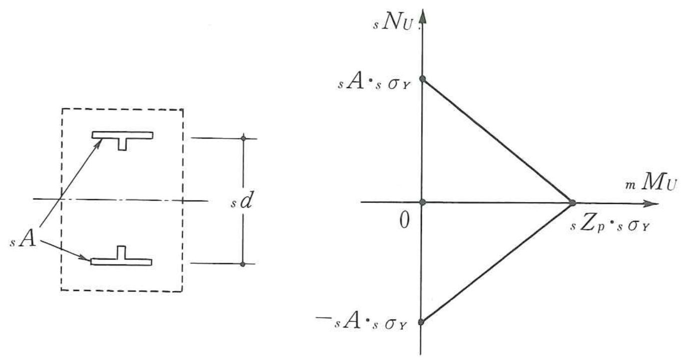

Fig. 32.5 Ultimate Flexural Strength of Open-web Steel

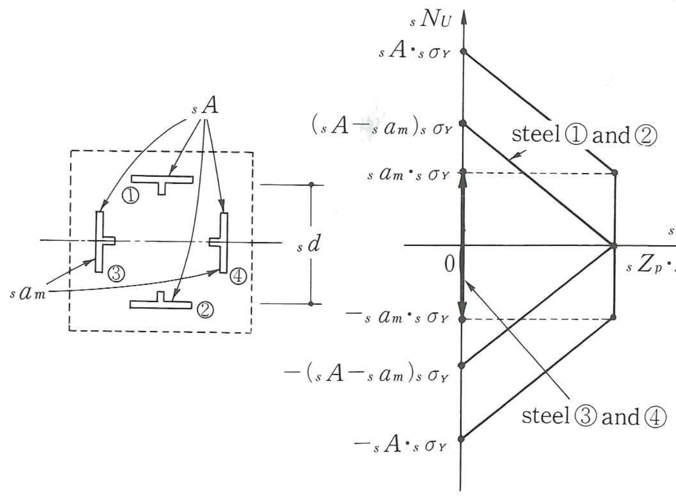

Fig. 32.6 Ultimate Flexural Strength of Cross-shaped Open-web Steel

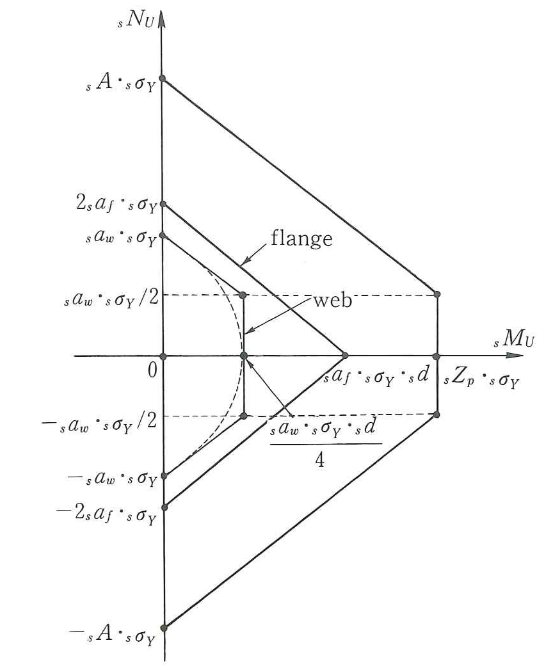

Fig. 32.7 Ultimate Flexural Strength of Full-web Steel (strong-axis bending)

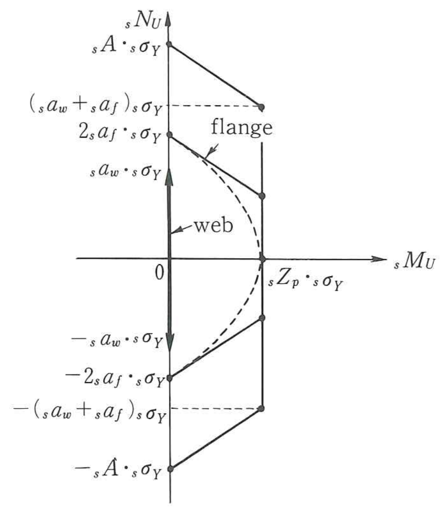

Fig. 32.8 Ultimate Flexural Strength of Full-web Steel (weak-axis bending)

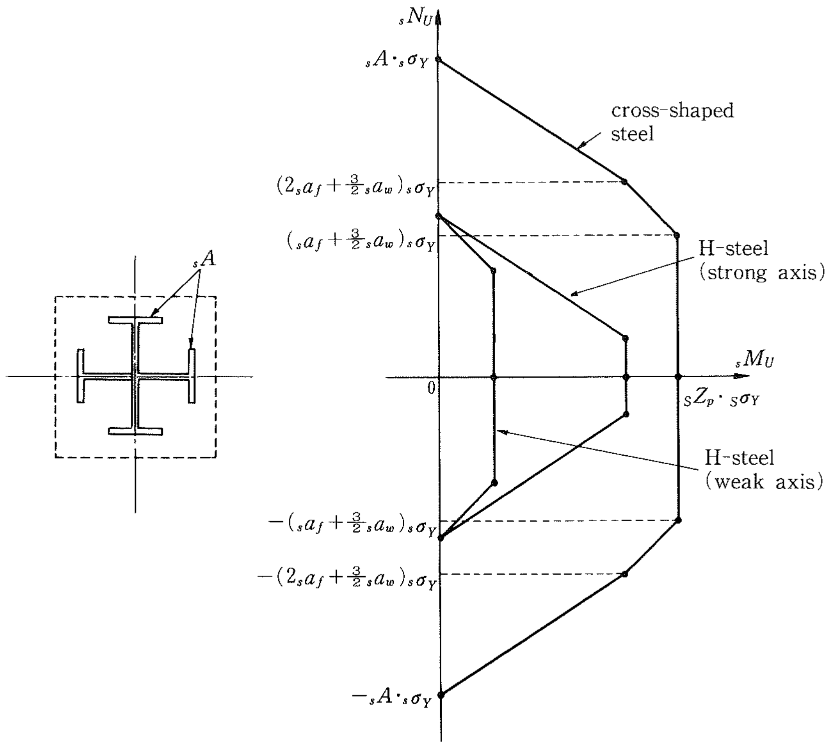

Fig. 32.9 Ultimate Flexural Strength of Cross-shaped Full-web Steel

Article 33 Ultimate Shear Strength of Members

1. The ultimate shear strength of a member is based on the method of superposition.

2. (3) The shear strength $_rQ_{sU1}$ is associated with the shear failure. The first term in the equation for $_rQ_{sU1}$ representing the resistance by concrete is 1.5 times that in the first expression of Eq. (44) for the allowable shear strength for the short-term loading condition, and it corresponds to the average value of the ultimate strength obtained in the shear tests of members. The first term in the equation for $_rQ_{sU2}$ associated with the shear bond failure is also 1.5 times that in the second expression of Eq. (44), and the former corresponds to the average of the test data while the latter to the lower bound (Fig. 18.1).

Article 34 Ultimate Shear Strength of Beam-to-Column Connections

The left hand side of Eq. (C20.4), which corresponds to the lower bound of the test data, multiplied by 1.2 gives the average value of the test data. Substituting $_jF_s = 0.12F_c$ and adding the contribution from the stirrups to the equation lead to Eq. (128).

Article 37 Ultimate Strength of Structural Walls

1. The ultimate flexural strength of an individual multi-story structural wall is obtained as listed in Table Cl in Appendix by the method of superposition (Fig. 37.1).

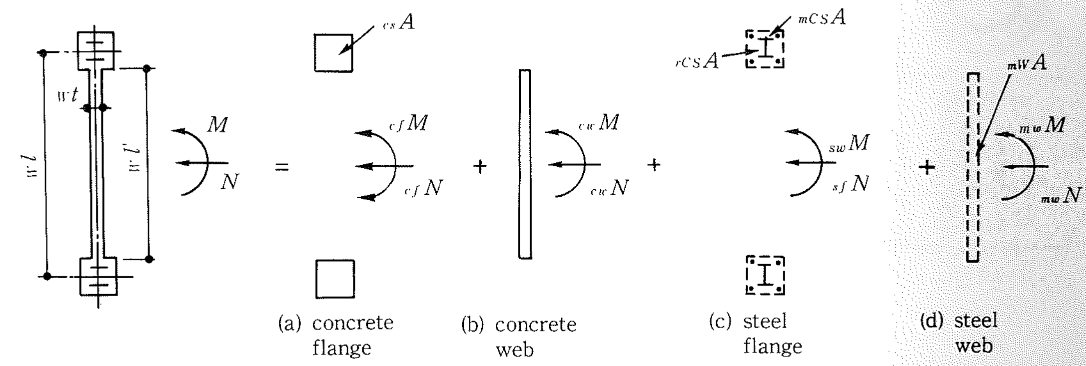

Fig. 37.1 Ultimate Flexural Strength of SRC Structural Wall

2. (1) The formula for the ultimate shear strength is similar to the formula for the allowable shear force specified in Article 25, Item 2, where the concrete strength is increased by 30 to 50%, and the allowable stress for steel is replaced by the yield stress. f_s' in Eq. (96) is replaced by F_s' in Eq. (138), and thus the shear strength of concrete is increased by 5/4 times.

Section 4 Calculation of Lateral Load Resisting Capacity

Article 38 General

1. A collapse mechanism of an SRC structure may form not only by the flexural yielding of members and the flexural yielding and rotation of structural walls but also by the shear yielding of members, connections and walls.

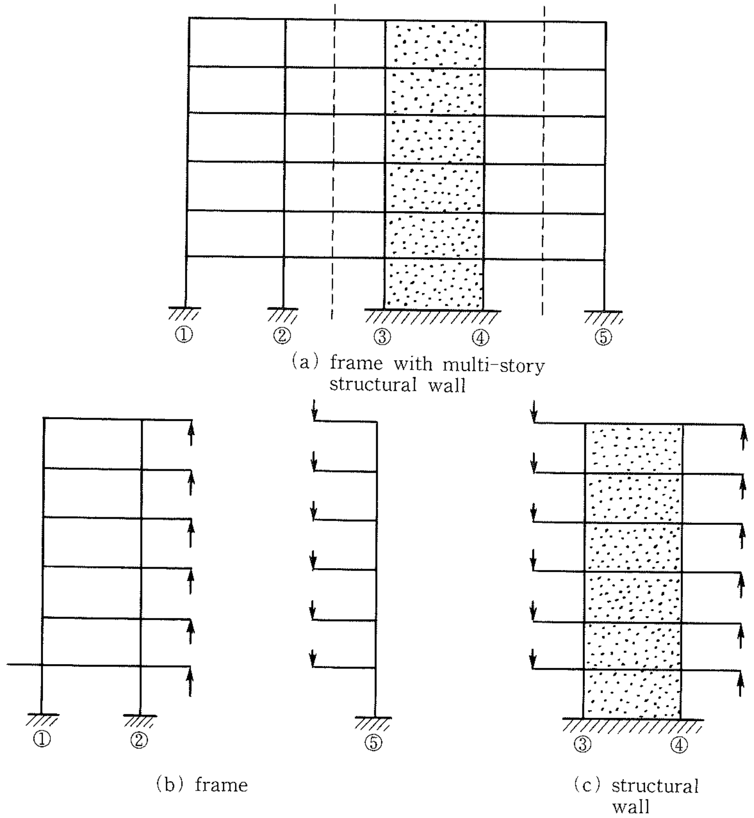

Fig. 38.1 Method of Separation of Frames from Multi-story Structural Wall

7. The lateral load resisting capacity of a frame with multi-story structural wall may be calculated by separating the frame into open frames and independent walls at intermediate points of beams connected to the walls, as shown in Fig. 38.1.

Chapter 6 Design of Composite Steel Tube and Concrete Structures

Section 2 Allowable Stress Design

Article 40 Calculation of Axial and Flexural Strengths of Columns

1. This article may apply to the cross sections shown in Fig. 40.1 in addition to those in Fig. 1.3.

(2) The flexural strength of a concrete-encased-and-filled steel tubular column is obtained by superposing the strengths of covering RC portion, filled concrete portion and steel tube, considering the consistency with the shear design. However, it is assumed that the filled concrete portion carries only axial force, since the flexural strength is smaller compared with the other two portions. Figure 40.2 outlines the procedure of superposition.

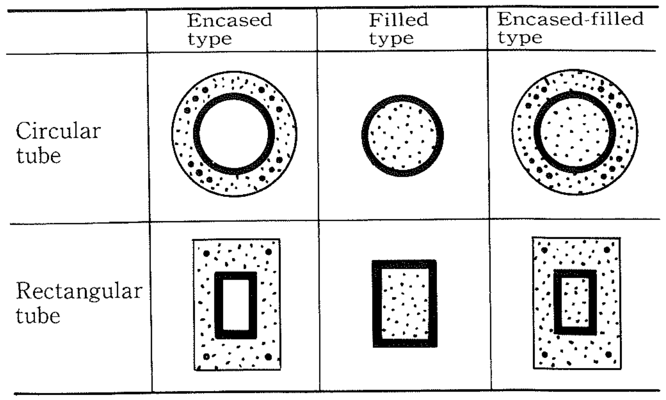

Fig. 40.1 Composite Steel Tube and Concrete Sections Covered by the Standards

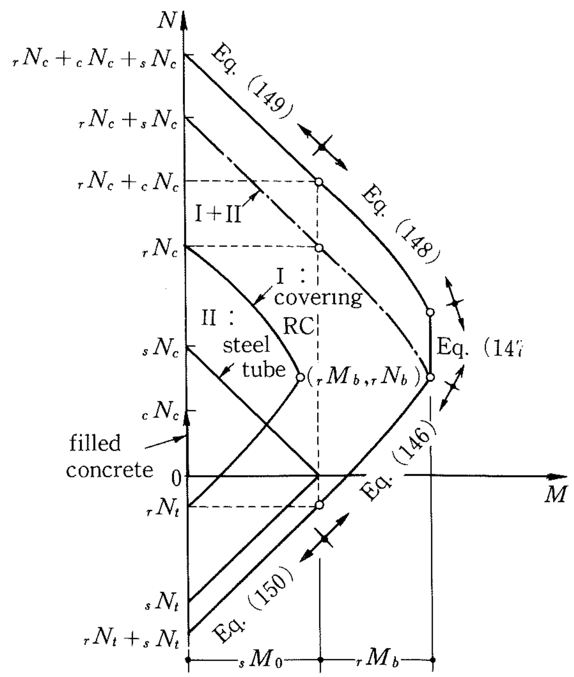

Fig. 40.2 Superposed Strength of Concrete-encased-and-filled Steel Tubular Column

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Article 41 Calculation of Shear Strength of Members

1. The behavior of a composite steel tube and concrete column under shear is similar to that of an SRC column using full-web steel, except for the filled concrete portion. The shear strength of the covering RC portion as shown in Fig. 41.1 is given by Eq. (C18.2). The shear force carried by the steel tube in (b) becomes the maximum when the ultimate flexural strength is reached at the column ends, unless the column fails in shear. The shear force carried by the filled concrete portion is obtained from the stress state assumed in (c), and the check for the shear stress of the filled concrete is not needed.

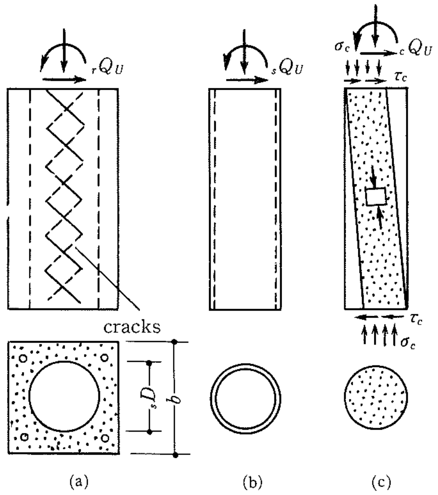

Fig. 41.1 Mechanisms of Shear Resistance of Composite Steel Tube and Concrete Column

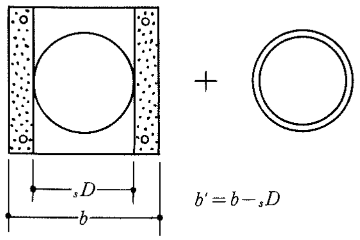

Fig. 41.2 Calculation Method for the Shear Strength of Concrete-encased Steel Tubular Column

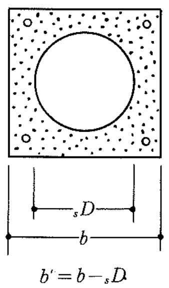

Fig. 41.3 Effective Width b' of Covering RC Portion

2. (1) Design concept for the allowable shear strength under the long-term loading condition is similar to that of an SRC column with full-web steel. Calculation for the allowable shear strength of both concrete-encased type and concrete-encased -and-filled type should be made by assuming conservatively that the rectangular section with width b' shown in Fig. 41.2 is effective. If Eq. (58) for ${}_rQ_{AL}$ of a general SRC column is applied to a composite steel tube and concrete column, the first expression of Eq. (59) governs, since the column width b is taken equal to b' as indicated above. Replacing b and α' in Eq. (58) by b' and α, respectively, leads to Eq. (167). Therefore, Eqs. (167) and (168) may not be directly applied to columns shown in Fig. 40.1.

(3) If Eq. (63) for the allowable shear strength of an SRC column under the short -term loading condition is applied to a concrete-encased type tubular column shown in Fig. 41.3, it becomes

$${}_rQ_{AS}=b'\cdot {}_rj(f_s+0.5{}_wp\cdot {}_wf_t) \qquad \text{(C 41.1)}$$

$${}_rQ_{AS}=2b'\cdot {}_rj(f_s+0.5{}_wp\cdot {}_wf_t) \qquad \text{(C 41.2)}$$

Since the width of the covering RC portion is b', Eq. (C41.1) controls the design and becomes identical to Eq. (171). However, the shear reinforcement ratio ${}_wp$ is based on the width b' as defined in Eq. (1), and the upper limit for ${}_wp$ is 1.2%, which is twice the value for a general SRC column.

Article 42 Bond between Steel Tube and Concrete

1. The compression force in a steel tube developed by shear at a beam end is normally transferred in a smooth manner to the covering RC portion outside the tube through horizontal stiffeners at the beam-to-column connection, which mechanically prevents the slippage between the tube and the concrete. However, there is no such a device inside the tube, thus the stress transfer from the tube to the filled concrete must be checked. Figure 42.1 illustrates the flexural strengths on an M-N interaction curve at several different heights of a concrete-filled steel tubular column. Suppose that the stresses at levels B and C above and below the beam 1 in Fig. 42.1 (a) are indicated by the points B and C in (b), respectively, the difference Δ_cN_1 between the axial forces carried by concrete at two levels indicated by ${}_cB$ and ${}_cC$ in (b) must be transferred from the tube to the concrete by bond. The transfer must be considered also for the difference in axial force Δ_cN_2 between two levels E and F for the beam 2. It may be considered that the bond strength distributed between center points of the upper and lower stories, that is, between the points A and D for the beam 1, is available for the axial force

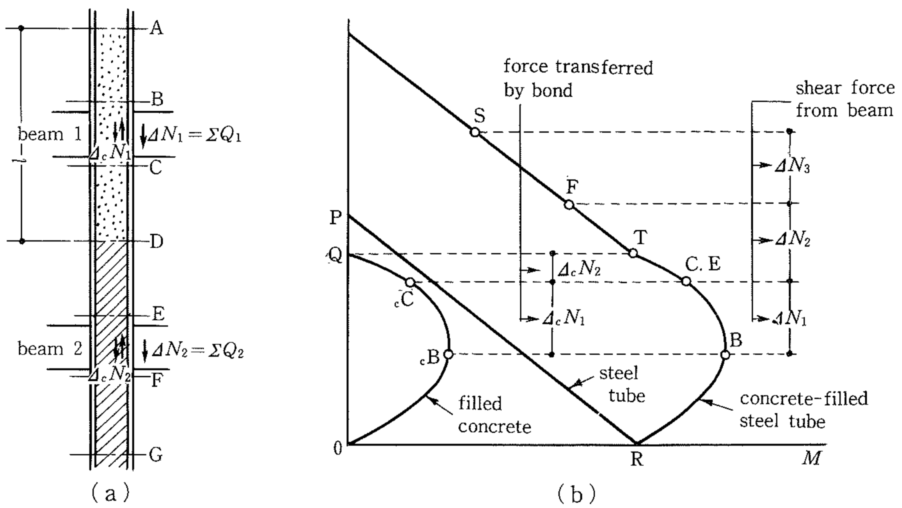

Fig. 42.1 Stress Transfer at Beam-to-column Connection of Concrete-filled Steel Tubular Column

transfer. If it is not enough, mechanical devices must be arranged inside the tube to prevent the slippage.

Article 43 Beam-to-Column Connections

The design formula for the beam-to-column connection of a concrete-encased type or a concrete-encased-and-filled type column has the same form as that for a general SRC structure, although some coefficients are a little different.

In the case of the connection with a concrete-filled type column, the shear strength is checked by the method of superposition for both the long-term and short-term loading conditions.

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Appendix Historical Review of Standards for Structural Calculation of Steel Reinforced Concrete Structures

Steel reinforced concrete (SRC) structural system has been used in the construction of high-rise buildings from the beginning of Taisho era (1910's). The structures were designed according to individual designer's engineering judgment since there was no structural standards available at that time. In 1951, Committee for SRC structures was organized under Structural Standard Committee in Architectural Institute of Japan to establish a design standard, and then the first edition of Standards for Structural Calculation of Steel Reinforced Concrete Structures and their Commentary (SRC Standards) were published. The Standards were characterized by the use of the method of superposed strengths to calculate the allowable strengths of members. The method has been used to the present edition of the SRC Standards. A small part was revised in 1963, and the use of light-weight concrete was added.

Tokachi-Oki earthquake of 1968, which caused brittle shear failure of RC columns, drew attention to the improvement of the design method for RC structures. The committee for SRC structures also started the revising work of the Standards with experimental studies toward more rigorous shear design, and published the second revision of the SRC Standards in 1975. Significant changes in the method of shear design were introduced in the Standards, and a new set of shear strength formulas and a concept of flexural yielding preceding shear failure were presented. Then, many SRC structures were subjected to intense Miyagiken-Oki earthquake of 1978, but only minor damages were observed in structural components, and the reliability of the SRC Standsrds was proved. The Revision of Enforcement Order of the Building Standard Law of Japan was proclaimed and enforced in 1980 and 1981, respectively. The third revision of the SRC Standards was published in 1987, showing the calculation method of the lateral load resisting capacity of SRC structures required by Enforcement Order.

As to composite steel tube and concrete structures, a subcommittee under the Committee for SRC Structures worked to develop the standards, and Standards for Structural Calculation of Composite Steel Tube and Concrete Structures and their Commentary were published in 1967, which showed design formulas based on the method of superposition employed in the SRC Standsrds. This was revised in 1980 to coordinate with the 1975 edition of SRC Standards, and the use of rectangular hollow section was newly included and a new set of design formulas for long columns were shown. In 1987, the standards for composite steel tube and concrete structures were absorbed to the third revised edition (present edition) of the SRC Standards.

AIJ Standards for Structural Calculation of
Steel Reinforced Concrete Structures (1987)

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