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Design Guideline for Building of High-Strength Reinforced Concrete Structures (Draft)

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Chapter 1 General

Zi-Liang Wu, Ker-Chun Lin

1.1 Scope

This design guideline is to apply to reinforced concrete building structures. Each material that is used in the structures and whose specified strength exceeds the upper limitations of the existing design code, "Design Specifications for Concrete Structures", shall be certificated in accordance with the provisions of Section 1.2 and all of the relevant requirements of this design guideline shall be satisfied.

Any material that is used in the structures and whose specified strength does not exceed the upper limitation of the existing design code, or any specifications not specified in this design guideline shall comply with the provisions of both "Design Specifications for Concrete Structures" and "Construction Specifications of RC Structures" promulgated by the Construction and Planning Agency Ministry of the Interior.

[Commentary]

This guideline should only apply to building structures due to the research evidences and literatures referred to this design guideline being based on the loading and structural characteristics of building structures. When designers apply this guideline to other types of structures, the applicability should be paid attention to.

The scope of this design guideline mainly focuses on the materials that specified strengths exceeds the strengths the existing design code, "Design

Specifications for Concrete Structures", (CPAMI, 2017) allowed. The upper limitations of specified strengths in this guideline are yield strengths of 690 MPa and 790 MPa for longitudinal and transverse reinforcements, and compressive strength of 100 MPa for concrete, respectively .

If only some materials in portions of a high-rise reinforced-concrete building are adopted the high strength materials mentioned previously, the design of these portions is applicable to this guideline. It is noted that the provisions of Section 1.2 should be considered. The other portions should be designed in accordance with the existing code.

1.2 Certifications for New Materials and New Constructions

The applicable scope of this design guideline is beyond that of the existing code "Concrete Structure Design Code". Therefore, a certification shall be approved in accordance with "Application Points for New Technologies, New Construction methods, New Equipment and New Materials of Building Approvals" promulgated by the Construction and Planning Agency Ministry of the Interior before a building is built according to this guideline.

[Commentary]

When applying any new materials, construction methods, technologies, or the related design methods which exceed the applicable scope of the existing code, some relevant tests or researches should be conducted, and a review procedure of applying to the Construction and Planning Agency Ministry of the Interior in accordance with the "Application Points for New Technologies, New Construction methods, New Equipment and New Materials of Building

Approvals" (CPAMI, 2010) to get a certification have to be done.

1.3 Reference Standards

> Unless otherwise specified, the standards cited in this design guideline are based on the Chinese National Standard (CNS) and supplemented by other relevant standards.

[Commentary]

The relevant specifications and standards cited in this guideline are listed in the references per section.

References

[1.1] Construction and Planning Agency Ministry of the Interior (CPAMI), 2010, "*Application Points for New Technologies, New Construction methods, New Equipment and New Materials of Building Approvals*," Taiwan. (in Chinese)

[1.2] Construction and Planning Agency Ministry of the Interior (CPAMI), 2017, "*Design Specifications for Concrete Structures*," Taiwan. (in Chinese)

[1.3] Construction and Planning Agency Ministry of the Interior (CPAMI), 2002, "*Construction Specifications of RC Structures*," Taiwan. (in Chinese)

Chapter 2 Materials

Wen-Cheng Liao, Ker-Chun Lin, Hung-Jen Lee, Kai-Ning Chi

2.1 Concrete

This design guideline is applicable to normalweight concrete with maximum of specified compressive strength f_c' of 100 MPa. The term of "high strength concrete" is used to refer to normalweight concrete with specified compressive strength f_c' between 70 MPa to 100 MPa. The material properties of high strength concrete shall comply with the requirements of this section. The matters not specified in this section shall comply with the relevant provisions of the "Design Specifications for Concrete Structures" and "Construction Specifications of RC Structures" promulgated by the Construction Department of the Ministry of the Interior.

[Commentary]

The high strength concrete of this design guideline is mainly applicable to normalweight concrete with compressive strength between 70 MPa and 100 MPa. This compressive strength range is verified by the relevant member test in this design guideline. In addition, the production capacity of domestic ready-mixed concrete suppliers is also considered. In "Construction Specifications of RC Structures" (CPAMI, 2002), high strength concrete is defined as concrete with compressive strength equal to or greater than 42 MPa, but the high strength concrete of this design guideline should still comply with its general provisions on the preparation and storage of concrete materials (cement, admixture, mixing water, and aggregates), production, transportation, pouring, curing, quality control, inspection and construction quality control.

High strength concrete should have suitable workability and other required properties. For high strength concrete with high flowability, it should also comply with corresponding special concrete regulations.

2.1.1 Constituent Materials

Cementitious materials and chemical admixtures used in high strength concrete shall conform to the relevant specifications as follows:

(1) Portland cement: CNS 61;

(2) Chemical admixtures for concrete: CNS 12283;

(3) Ground granulated blast-furnace slag for use in concrete and mortars: CNS 12549;

(4) Coal fly ash and raw or calcined natural pozzolan for use as a mineral admixture in concrete: CNS 3036;

(5) Silica fume used in cementitious mixtures: CNS 15648.

The properties and amount of the admixtures used should be recorded to the authority for approval. Aggregates of high strength concrete shall conform to the following specifications:

(6) Concrete aggregates: CNS 1240.

The nominal maximum size of the coarse aggregates should not be too large, and the particles are hard and well graded.

[Commentary]

For cement used in high-strength concrete, in addition to satisfying the relevant standards, it is advisable to select cement with higher fineness. Due to relatively low water-to-cementitious material ratio in high strength concrete, workability and excessive autogenous shrinkage should be considered. Therefore, it is allowed to add chemical admixture (retarding, accelerating, water reducing, expansion admixtures) and mineral admixture (fly ash, granulated blast-furnace slag, silica fume), but all admixtures must be verified in advance and submitted for approval. The fine aggregate with higher fineness modulus is also preferable because of high powder volume in high strength concrete mixtures. The fineness modulus of fine aggregates should be between 2.7 and 3.1. In order to reduce the influence of the interfacial transition zone (ITZ) between the matrix and the coarse aggregate, the coarse aggregate are required to be clean with low coatings of clay (the content is not more than 1%), it is advisable to use coarser aggregate with a smaller particle size, preferably below 12.5 mm. The abrasion obtained by test for resistance to degradation of coarse aggregate by abrasion and impact in the Los Angeles machine should be less than 25%, and the weighted percentage loss of the test for soundness of aggregate by use of sodium sulfate should be less than 5%. Table C2-1 is an example of the coarse aggregate required properties of high strength concrete.

Table C2-1 Required properties of coarse aggregate in high strength concrete.

Required properties		
SSD Specific Gravity		*>2.5*
Absorption and surface moisture		*<1.5%*
Abrasion and degradation (CNS 490, 2009)		*<25%*
Weighted percentage loss of test for Soundness of Aggregate by Use of Sodium Sulfate (CNS1167, 1995)		*<5%*
flat particles, elongated particles, or flat and elongated particles contents in coarse aggregate (CNS 15171, 2008)		*<10%*
Sieve Analysis-Cumulative passing percentage (%)	*37.5 mm (3/2")*	*--*
	25 mm (1")	*100*
	19 mm (3/4")	*100*
	12.5 mm (1/2")	*90-100*
	9.5 mm (3/8")	*40-70*
	4.75 mm (No.4)	*0-15*
	2.36 mm (No.8)	*0-5*
Alkali-Silica Reaction		*w/o harmful substances*

2.1.2 Preparation and Storage of Materials

The raw materials of high strength concrete shall be separated and stored without being mixed, and the storage silos shall be marked for identification. Any material should be properly stored and should not be used if it is damaged, contaminated or deteriorated. The aggregate should be stored in separate closed silos to avoid mixing of different sizes of aggregates or other materials for better control of the temperature and water content.

[Commentary]

The quality of aggregates affects the mechanical properties of high strength concrete. Its mud content and water content directly affect the strength of concrete. Therefore, it should be stored in separate closed silos to avoid mixing with different sizes of aggregates or other materials. In addition to quality control, it is also convenient for the sampling inspection.

2.1.3 Mix Design and Trial Mix

The workability of high strength concrete can be adjusted by adding chemical admixtures due to its low water-to-cementitious materials ratio. The fresh properties shall be tested according to relevant specifications to determine the mix proportions. In order to enhance the properties of the transition zone between the matrix and aggregates, it is recommended to adopt a two-step mixing process. When the supplier conducts the trial mixing, the strength development and elastic modulus of the 7, 28, 56, and 91-day should be measured.

[Commentary]

The water-to-cementitious materials ratio of high strength concrete should be below 0.30, so it is necessary to adjust the workability by adding chemical admixtures. In order to strengthen the transition zone between the matrix and the aggregate, it is recommended to adopt a two-step mixing process, which is to stir the cement and some of the mixing water to form a paste, then add the coarse aggregate to continue stirring, and then add the fine aggregate and the remaining mixing water. The requirements for quality stability of high strength concrete is more stringent, so it is necessary to carry out trial mix to confirm the quality. In addition to the laboratory trial batches, it is also required field trial mix at the construction site to evaluate the workability, slump loss, initial and final setting time, and compressive strength development. In addition to recording the strength development, due to higher paste volume and less amount of coarse aggregate, it is recommended to simultaneously measure the elastic modulus for design and reference.

2.1.4 Quality Control

Within six months prior to the production of high strength concrete, the contractor shall prepare a concrete quality control plan to specify the source of materials, material quality control, mix design, construction plan, construction technology, machinery and facility, quality control equipment, organization and operation methods, and backup plan. The quality control plan shall be proposed in detail and approved to ensure the quality of concrete materials and construction.

2.1.5 Modulus of Elasticity

Modulus of elasticity, E_c, for high strength concrete shall be permitted to be calculated as follows:

$$E_c = k_{sf} \times \left[23100 \times \left(\frac{w_c}{2380} \right)^{1.5} \times \left(\frac{f_c^{'}}{70} \right)^{0.5} + 7300 \right] \quad \text{(MPa)} \qquad (2\text{-}1)$$

where w_c is unit weight of concrete (kg/m^3), $f_c^{'}$ is specific compressive strength, and k_{sf} is modification coefficient for the silica fume. For the average value of unit weight of high strength concrete of 2380 kg/m^3, E_c shall be permitted to be taken as:

$$E_c = k_{sf} \times \left[2765 \times \left(f_c^{'} \right)^{0.5} + 7300 \right] \quad \text{(MPa)} \qquad (2\text{-}2)$$

k_{sf} shall be calculated as

$$k_{sf} = 1 - 0.78 \times S, \quad S \leq 25\% \qquad (2\text{-}3)$$

where S is the weight ratio of silica fume to total cementitious materials.

[Commentary]

The modulus of elasticity is an important property of concrete. It is also a key parameter for engineers to calculate deflection and stiffness. Shrinkage and creep of concrete is related to modulus of elasticity as well. According to Taiwan's current code or ACI 363 (ACI, 2010), the formulas are obviously overestimated and cannot fully reflect the low elastic modulus of high strength concrete in Taiwan.

The elasticity of modulus of high strength concrete in this design guideline is proposed by Liao (Liao et al, 2017), which collects the 449 test data of high strength concrete test from colleges and industries (average unit weight of high strength concrete is 2,380 kg/m^3). This proposed formula is also applicable to high strength concretes with different ages. The recommended formula of this design guideline is compared with the test results as shown in Figure C2-1.

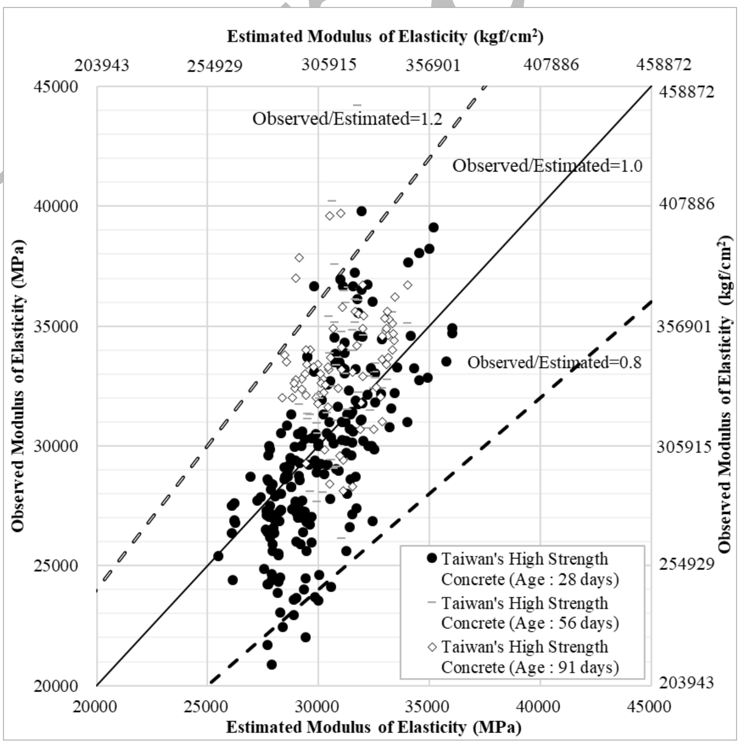

Fig. C2-1 Comparisons of predictions calculated by the proposed formula to the test results. (Liao et al, 2017)

Similar observation in this proposed formula can be also found in the research regarding elasticity of modulus of high strength concrete in Japan (Tomosawa, 1990), which is adding silica fume will increase the strength of concrete, but the modulus of elasticity will not increase proportionally. Therefore, when predicting the elasticity of modulus of high strength concrete containing silica fume, a modification factor k_{sf} should be applied to reflect this phenomenon. The relationship between the residuals (predicted value minus the experimental value) with/without involving k_{sf} and the content of silica fume are shown in Figure C2-2.

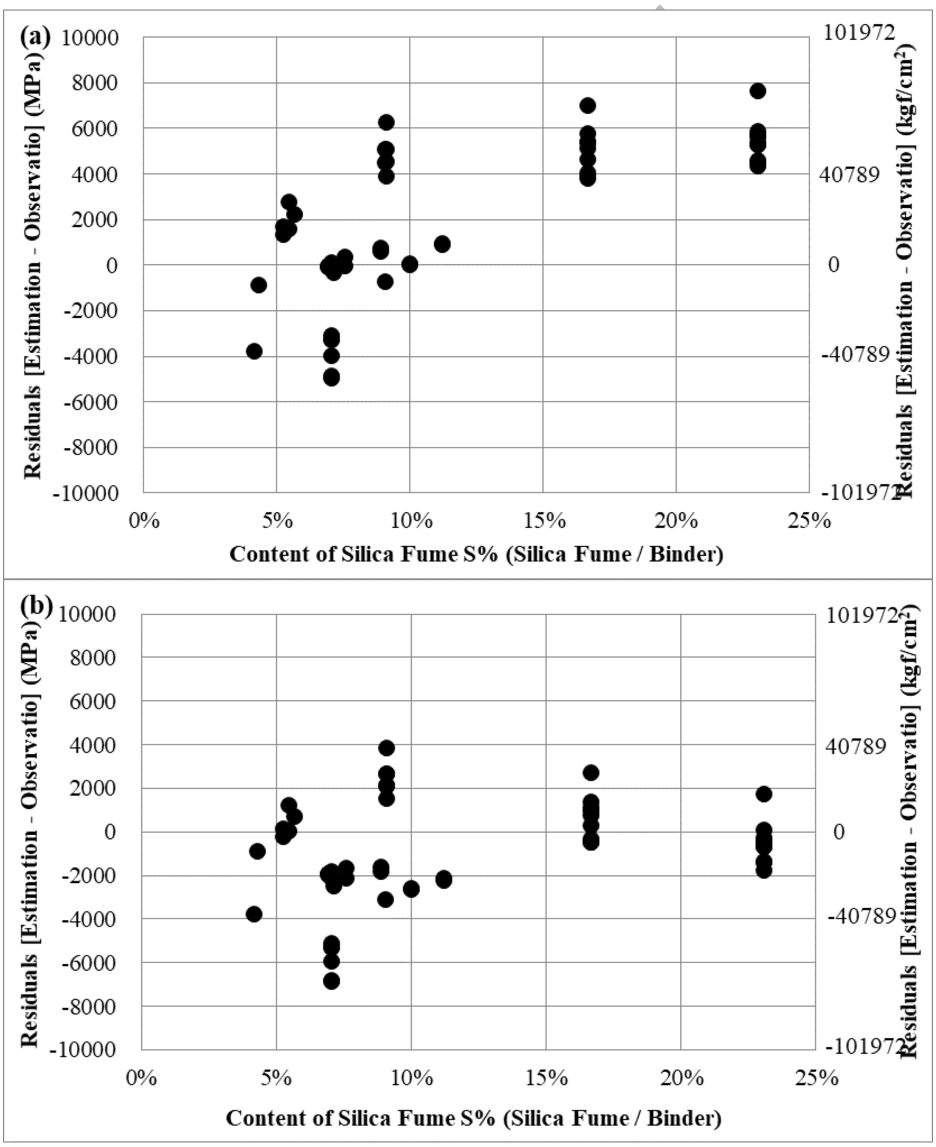

Fig. C2-2 Comparisons of residuals by the proposed prediction formula (a)with and (b) without involving k_{sf}. (Liao et al, 2017)

2.1.6 Approval for Specific Applications

For specific applications of high strength concrete, if other properties are required, such as high flowability, non-shrinkage for concrete filled tube columns, or adding fibers, it shall comply with relevant regulations.

[Commentary]

If other properties are required for specific applications, the high strength concrete should comply with relevant regulations. When the high strength concrete is applied to concrete filled tube (CFT) columns, due to its high paste volume and low water-to-cementitious materials ratio, noticeable autogenous shrinkage and plastic settlement are often observed. Therefore, tests of the 91-day expansion rate, 24-hour bleeding rate and the 24-hour plastic settlement should be carried out according to CNS C14603 (CNS, 2001), CNS 1235 (CNS, 1998), JASS 5T-503:2009 (JASS, 2009), respectively. The relevant quality requirements should be specified and recorded in the contract. The expansion admixtures for concrete (non-shrinkage cement chemical admixtures) should comply with the provisions of CNS 10641 (CNS, 1983) or ASTM C845 (ASTM, 2012). Table C2-2 is an example of the requirements for the non-shrinkage self-consolidating concrete in CFT columns.

Table C2-2 Requirements of non-shrinkage concrete in CFT columns.

Specific property	*Requirement*	*Test Provision*
91-day expansion rate	*(0.04±0.02) %*	*CNS C14603*
24-hour bleeding rate	*<0.02* cm^3/cm^2	*CNS 1235*
24-hour plastic settlement	*<2 mm*	*JASS 5T-503:2009*

2.2 Reinforcement

The high-strength steel reinforcement in this design guideline refers to those with specified yield strength exceeding the upper limit stipulated in the "Design Specifications for Concrete Structures". The properties of steel reinforcement shall be in accordance with 2.2.1. Devices for mechanical splice and anchorage shall conform to 2.2.2.

[Commentary]

The high-strength steel reinforcement in this design guideline refers to those with specified yield strength exceeding the upper limit stipulated in the "Design Specifications for Concrete Structures" (CPAMI ,2017). The properties of steel reinforcement provided in 2.2.1 also comply with the CNS 560 (CNS, 2018). Due to SD 790 not including in CNS 560, SD 790 reinforcement should conform to requirements of "High-Strength Steel Bars for Concrete Reinforcement (SD 550/685/785)" (TCI, 2014a) by Taiwan Concrete Institute. The bars of SD 790 only apply to the transverse reinforcement which is used for confined or resisting shear forces.

The purpose for the devices of the mechanical splice and anchorage shall conform to 2.2.2 which are corresponding to the high-strength steel reinforcement is transferring the force between rebar and connection. Additionally, the performance assessment of mechanical splice and anchorage performance for connected parts shall conform to requirements of "Guidelines for Performance Evaluation of Mechanical Splices for High-Strength Steel Reinforcing Bars" (TCI, 2014b) or "Specification for Headed Steel Bars for Concrete Reinforcement" (TCI, 2014c), respectively, are provided in 2.2.2.

2.2.1 Steel Reinforcement Properties

The required mechanical properties and geometries for the high-strength steel reinforcement in this design guideline are provided as follows:

(1) Bars are of four minimum yield strength levels: designated as SD 490W, SD 550W, SD 690 and SD 790, respectively. The mechanical properties of reinforcement are given in Table 2-1. SD 690 bars shall not be welded, and shall not be anchored by hooks or welded devices for bar size greater than or equal to D19. Additionally, the bars of SD 790 shall not apply to longitudinal reinforcement, only apply to transverse reinforcement for confinement and resisting shear. The nominal elastic modulus E_s of high-strength reinforcement addressed in this guideline is 200 GPa.

(2) For deformed and threaded bars designed as flexural reinforcement where bond strength is critical to transfer the force, the ratio of average rib height to the average rib spacing shall not be less than 0.12 and 0.17, respectively.

Table 2-1 Mechanical properties of the steel reinforcement.

Category	Grade	Mechanical Properties							
		Yield Strength[(a)] N/mm²	Tensile Strength N/mm²	Actual Tensile Strength / Actual Yield Strength	Test Specimen[(c)]	Elongation %	Bending Requirements		
							Bend Angle	Bend Diameter	
Deformed Bars Threaded Bars	SD 490W	490~540	615↑	1.25↑	No.2	13↑	180°	D16↓	$3d_b$
								D19~D25	$4d_b$
					14A	14↑		D29~D36	$6d_b$
								D39↑	$8d_b$
	SD 550W	550~675	690↑	1.25↑	No.2	12↑	180°	D16↓	$3.5d$
								D19~D25	$5d_b$
					14A	13↑		D29~D36	$7d_b$
								D39↑	$9d_b$
	SD 690	690~815	860↑	1.15[(b)] ↑	No.2	10↑	180°	D16↓	$3.5d$
								D19~D25	$5d_b$
					14A	10↑		D29~D36	$7d_b$
							90°	D39↑	$9d_b$
	SD 790	790↑	930↑	—	No.2 14A	8↑[(d)]	180°	D16↓	$3d_b$

Note: Except SD 690 steel, the specified elongations of the deformed bars and threaded bars shown in this table are applicable to the designations of D36 or smaller and they should be instead of deducting 2% from the values shown in Table 2-1 for the designations of D39 or greater.

[a] Where the steel does not have a well-defined yield point, the yield strength shall be determined by the 0.2% offset method as described in CNS 2111 (CNS, 1996).

[b] Purchasers can demand that the lower limit value of the ratio of actual tensile strength to actual yield strength is 1.25.

[c] Steel coupons for tensile test shall conform to requirements of CNS 2112 (CNS, 2005).

[d] The welded point of SD 790 steel reinforcement shall not be bent during construction and testing.

[Commentary]

The specification of SD 490W, SD 550W and SD 690 steel reinforcement in Table 2-1 shall conform to requirements of CNS 560. Additionally, SD 790 steel reinforcement shall conform to requirements of “High-Strength Steel Bars for Concrete Reinforcement (SD 550/685/785)” by Taiwan Concrete Institute.

The chemical composition of SD 490W, SD 550W, SD 690 and SD 790 reinforcement shall conform to requirements of Table C2-3. According to AWS D1.4 (AWS, 2011), the carbon equivalent (C.E.) of SD 490W and SD 550W shall not be greater than 0.55% for weldability.

SD 690 steel reinforcement should apply to longitudinal reinforcement. Due to the higher strength and no upper limit of carbon equivalent for SD 690 reinforcement, the rebar is not good for cold bending or welding. Therefore, SD 690 reinforcement is not recommended for adopting standard hook, welded mechanical splice and anchorage, or assemblies.

SD 790 steel reinforcement should apply to transverse reinforcement. If the closed-hoop stirrup manufacture by resistance welding, the processing of stirrup shall be in accordance with the appropriate procedure in factory. The maximum diameter at welded point should be between 1.7 and 2.2 times the nominal diameter of bars, but the eccentric distance of axis line should not be larger than 10% the nominal diameter of bars.

Table C2-3 Chemical composition of the steel reinforcement.

Category	*Grade*	*Chemical Composition (%)*					
		C	M_n	*P*	*S*	*Si*	*C.E*[(a)]
Deformed Bars (Fig. C2-3)	*SD 490W* *SD 550W*	*0.33↓*	*1.56↓*	*0.043↓*	*0.053↓*	*0.55↓*	*0.55↓*
Threaded Bar (Fig. C2-4)	*SD 690*	—	—	*0.075↓*	—	—	—
	SD 790	*0.50↓*	*1.80↓*	*0.030↓*	*0.030↓*	*1.50↓*	—
Note: [(a)] *C.E.(carbon equivalent)* $=[C+\frac{\mathrm{Mn}}{6}+\frac{\mathrm{Cu}}{40}+\frac{\mathrm{Ni}}{20}+\frac{\mathrm{Cr}}{10}-\frac{\mathrm{Mo}}{50}-\frac{\mathrm{V}}{10}]\%$							

The required length for lap splice increases as the reinforcement strength increases. It is impractical nor economic to have a long lap splice length due to steel congestion and poor concrete flowability in the splice regions. Therefore, it is recommended to use mechanical splice to replace the lap splice for high-strength steel reinforcement. It is also recommended to use mechanical anchorage to replace the standard hook for high-strength steel. In Japan, the grout-filled sleeves and grout-filled anchorage devices that are widely used in practices have shown satisfactory mechanical properties.

Requirements of rib height and rib spacing for the steel reinforcement are specified in the current CNS 560, with no specification about the ratio of the rib height to the rib spacing that is also critical to the bond strength between steel reinforcement and concrete. To ensure that the design equation in the code (CPAMI, 2017) work properly for the high-strength steel reinforcement, the ratio of average rib height to average rib spacing of the deformed bar should be 12% or greater as per ACI Committee 408 (ACI, 2003); the ratio of average rib height to average rib spacing of the threaded bar should be 17% or greater as per Lin's results (Lin, 2018). Geometry requirements for the deformed and threaded bars are shown in Table C2-4.

Table C2-4 Designation, unit mass, nominal dimensions and geometry requirements for deformed bars and threaded bars.

Desig-nation	Nom. No.	Unit Mass	Nominal Diameter	Nominal Cross-Section Area	Nominal Perimeter	Average Dimensions of Ribs			
						Spacing of Ribs Max.	Height of Ribs a		Single Gap Max.
	#	W (kg/m)	d_b (mm)	S (mm^2)	ℓ (mm)	p (mm)	Minimum (mm)	Maximum (mm)	b (mm)
D10	3	0.560	9.53	71.33	30	6.7 (4.8)	0.4	0.8	3.7
D13	4	0.994	12.7	126.7	40	8.9 (6.4)	0.5	1.0	5.0
D16	5	1.56	15.9	198.6	50	11.1 (8.0)	0.7	1.4	6.2
D19	6	2.25	19.1	286.5	60	13.3 (9.6)	1.0	2.0	7.5
D22	7	3.04	22.2	387.1	70	15.6 (11.1)	1.1	2.2	8.7
D25	8	3.98	25.4	506.7	80	17.8 (12.7)	1.3	2.6	10.0
D29	9	5.08	28.7	646.9	90	20.1 (14.4)	1.4	2.8	11.3
D32	10	6.39	32.2	814.3	101	22.6 (16.1)	1.6	3.2	12.6
D36	11	7.90	35.8	1007	113	25.1 (17.9)	1.8	3.6	14.1
D39	12	9.57	39.4	1219	124	27.6 (19.7)	2.0	4.0	15.5
D43	14	11.4	43.0	1452	135	30.1 (21.5)	2.1	4.2	16.9
D50	16	15.5	50.2	1979	158	35.1 (25.1)	2.5	5.0	19.7
D57	18	20.2	57.3	2579	180	40.1 (28.7)	2.9	5.8	22.5

The values shown in ( ) is the maximum spacing of ribs for the threaded bar.

Note 1. The calculation rules for the values given in Table C2-4 are based on the following formulas. The values shall be rounded off in accordance with CNS 2925 (CNS, 1968). However, the actual measured values shall conform to this table.

Note 2. Unit mass (kg/m), calculated by the following formula and rounded to 3 significant digits.

$$W=0.00785\times S$$

Note 3. Nominal section area (mm^2), calculated by the following formula and rounded to 4 significant digits.

$$S=0.7854\times d_b$$

Note 4. Nominal perimeter (mm), calculated by the following formula and rounded to the nearest integer.

$$\ell=3.142\times d_b$$

Note 5. Maximum average spacing of rib (mm), calculated by the following formula and rounded to 1 decimal place.

deformed bar, $p=0.7\times d$

threaded bar, $p=0.5\times d$

Note 6. Minimum average height of rib (mm), calculated by the following formula and rounded to 1 decimal place.

D10 to D13 : $\frac{4}{100}\times d$

D16 : $\frac{4.5}{100}\times d$

D19 to D57 : $\frac{5}{100}\times d$

Note 7. Maximum single gap (mm), calculated by the following formula and rounded to 1 decimal place.

$$b=0.125\times\ell$$

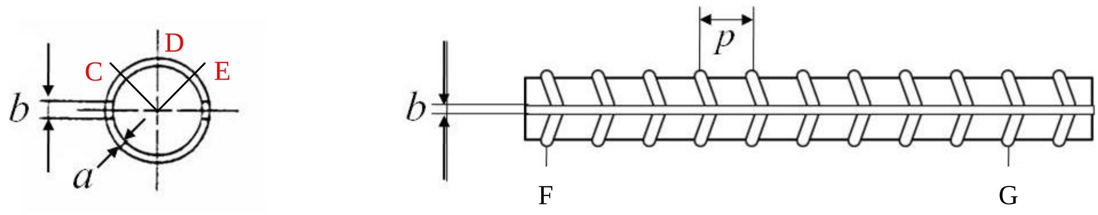

Figure C2-3 Examples for deformed bars. (CNS, 2018)

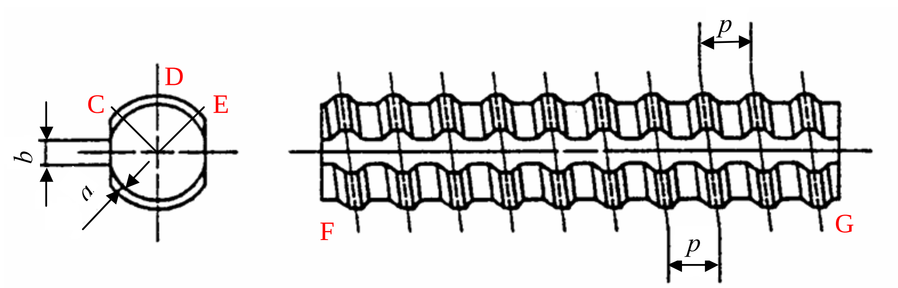

Figure C2-4 Examples for threaded bars. (CNS, 2018)

2.2.2 Mechanical Splice and Anchorage Devices

Mechanical splice and anchorage devices should be provided based on steel strength and their mechanical properties that meet the deformation demands at the location where the two steel reinforcement joints or the steel reinforcement is terminated.

[Commentary]

Mechanical splices can be classified as Type SA and Type B based on their mechanical properties as specified in "Guidelines for Performance Evaluation of Mechanical Splices for High-Strength Steel Reinforcing Bars" (TCI, 2014b) by Taiwan Concrete Institute.

Please refer to the "Specification for Headed Steel Bars for Concrete Reinforcement" (TCI, 2014c) by Taiwan Concrete Institute for the assessment of *mechanical anchorage devices.*

2.3 Material Strength Ratio of Concrete to Reinforcement

The specified compressive strength of concrete shall be at least 1/15 of the specified yield strength of the longitudinal reinforcement.

[Commentary]

The minimum ratio of the specified concrete compressive strength to the specified reinforcement yield strength which is provided in current "Design Specifications for Concrete Structures" (CPAMI, 2017) is 1/20. Generally, the ratio is not lower than 1/15 in actual engineering. Based on the available experimental data, it is recommended that the specified compressive strength of concrete should be at least 1/15 of the specified yield strength of the longitudinal reinforcement in this design guideline.

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Chapter 3 Beams

Min-Yuan Cheng, Yung-Chih Wang, Chien-Kuo Chiu

3.1 Scope

This chapter applies to beams designed primarily to resist flexure and shear forces. Beam shall satisfy (1) through (3):

(1) The factor axial compressive force should not exceed $0.1A_g f_c'$, where A_g and f_c' is the gross section area and specified concrete strength of the beam, respectively.

(2) Clear span ℓ_n should be at least $4d$.

(3) Width b_w should be at least the less of $0.3h$ and 250 mm.

Items not specified in this chapter shall conform to requirements of "Design Specifications for Concrete Structures" and "Construction Specifications of RC Structures".

[Commentary]

Factored axial compressive force is limited such that the interaction between axial force and moment may be negligible. Dimensional limits intend to prevent short-beam effects.

3.2 Flexure Strength

3.2.1 Nominal Flexural Strength

Nominal flexure strength of the beam is determined assuming that strain across a cross section is linearly distributed and the maximum strain at the extreme concrete compressive fiber equal to 0.003. Concrete compressive stress can be evaluated using equivalent rectangular stress block with concrete stress of $\alpha_1 f_c^{'}$ uniformly distributed along the height of $\beta_1 c$, represented in Fig. 3-1. In which, c is the distance from the fiber of maximum compressive strain to the neutral axis, $f_c^{'}$ is the specified concrete strength, α_1 and β_1 should be calculated by Eq. (3-1) and Eq. (3-2). Stress-strain (f_s-ε_s) relationship for deformed reinforcement can be idealized in accordance with Eq. (3-3), where f_y is the specified yield strength of reinforcement and ε_y obtained by f_y/E_s is the yield strain of reinforcement.

$$0.7 \le \alpha_1 = 0.85 - 0.0022\left(f_c^{'} - 55 \right) \le 0.85 \tag{3-1}$$

$$\beta_1 = \begin{cases} 0.85 & f_c^{'} \le 27.5\ MPa \\ 0.85 - 0.0073(f_c^{'} - 27.5) & 27.5\ MPa \le f_c^{'} \le 55\ MPa \\ 0.65 & f_c^{'} \ge 55\ MPa \end{cases} \tag{3-2}$$

$$f_s = \begin{cases} E_s \varepsilon_s, & when\ \ \varepsilon_s \le \varepsilon_y \\ f_y, & when\ \ \varepsilon_s > \varepsilon_y \end{cases} \tag{3-3}$$

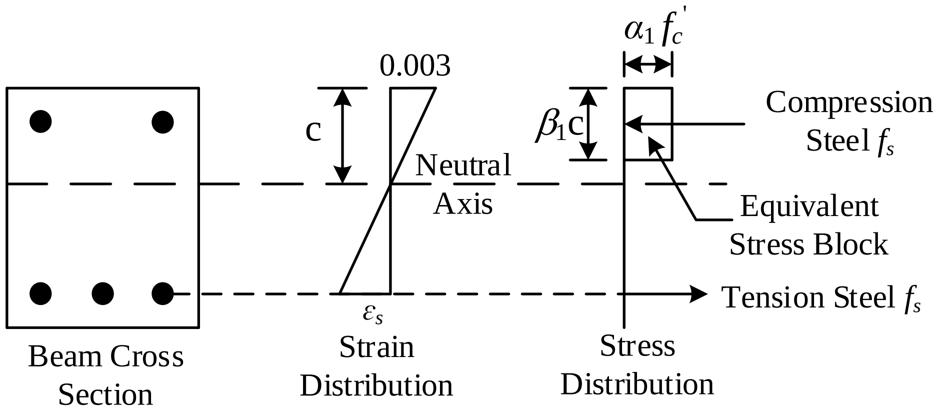

Fig. 3-1 Assumption of strain and stress distribution for calculation of sectional bending strength.

[Commentary]

Calculation of the beam nominal flexural strength assumed stress and strain distribution shown in Fig. 3-1. The corresponding maximum strain at the extreme concrete compressive fiber is assumed as 0.003. Concrete compressive force can be determined using equivalent rectangular stress block. Values of α_1 *and* β_1 *are based on results of ACI ITG-4.3R (ACI Innovative Task Group 4, 2007). A perfect elastic-plastic stress-strain behavior is suggested by Eq. (3-3) for the steel reinforcement.*

3.2.2 Strength Reduction Factors

The strength reduction factors (Φ) for nominal flexural strength are determined by Table 3-1. Where ε_t is the net tensile strain in extreme layer of longitudinal tension reinforcement at nominal flexural strength; ε_y is the yiled strain of the reinforcement which can be calculated by f_y/E_s.

Table 3-1 The strength reduction factors for nominal flexural strength.

ε_t	Classification	Φ
$\varepsilon_t \leq \varepsilon_y$	Compression-controlled	0.65
$\varepsilon_y < \varepsilon_t < \varepsilon_y + 0.003$	Transition	$0.65 + 0.25\dfrac{\left(\varepsilon_t - \varepsilon_y\right)}{0.003}$
$\varepsilon_t \geq \varepsilon_y + 0.003$	Tension-controlled	0.90

[Commentary]

This section specifies the strength reduction factors for nominal flexural strength. Compression-controlled section refers to those with the net tensile strain in the extreme tension reinforcement at nominal strength less or equal than ε_y. Tension-controlled section refers to those with the net tensile strain in the extreme tension reinforcement at nominal strength greater than the suggested value that varies linearly between 0.005 and 0.007 as the steel specified yield stress increases from 420 MPa to 690 MPa. The strain limit for tension-controlled section has been suggested as 0.008 and 0.009 by Shahrooz et al. (2013) and ACI ITG 6 (2010), respectively, for steel reinforcement in compliance with ASTM 1035 (ASTM A1035/A1035M, 2011). Based on test results of beams subjected to monotonically increased four-point load, Giduqio et al. (2015) report that the two specimens using SD685 steel with the net tensile strain in the extreme tension reinforcement of around 0.0068 at nominal flexural strength exhibited displacement ductility of 4.21 and 4.82, respectively. The two specimens using SD 420 with the net tensile strain in the extreme tension reinforcement of around 0.0045 at nominal flexural strength exhibited displacement ductility of 3.82 and 3.50, respectively. As a result, this design guideline suggests a strain limit of 0.007 for tension-controlled section using SD 690 steel.

3.3 Shear Strength

3.3.1 Shear Demand

Shear demand of the beam, V_e, should be determined as the maximum possible shear developed between the faces of the joints. It is permitted to be estimated assuming that moments of opposite sign corresponding to probable flexural strength, M_{pr}, act at the joint faces and that the beam is loaded with the factored tributary gravity load along its span.

[Commentary]

The design philosophy of strong-column-weak-beam and anticipation of plastic hinges developed at the beam ends require beam shear demand to be estimated based on flexure strength that may be developed at the ends of the beam and the supported gravity load rather than on factored shear forces indicated by structural analysis. The way to determine shear demand of the beam is illustrated in Fig. C3-1.

Because the actual yield strength of the longitudinal reinforcement may exceed the specified yield strength and because strain hardening of the reinforcement is likely to take place at a joint subjected to large rotations, the probable flexure strength, M_{pr}, *per ACI 318-14 (ACI, 2014) is determined using a stress of at least* $\alpha_o f_y$ (α_o = 1.25) *in the longitudinal reinforcement with specified yield stress of 420 MPa or less. Based on test results of beams with identical cross section of 400×700 mm, Cheng et al. (2017) indicate that peak strengths of specimens using SD 690 longitudinal reinforcement can be satisfactorily estimated using* $1.20f_y$. *In this study, tensile strain calculated in the outmost-layered tension reinforcement at nominal flexure strength is between*

0.008~0.020.

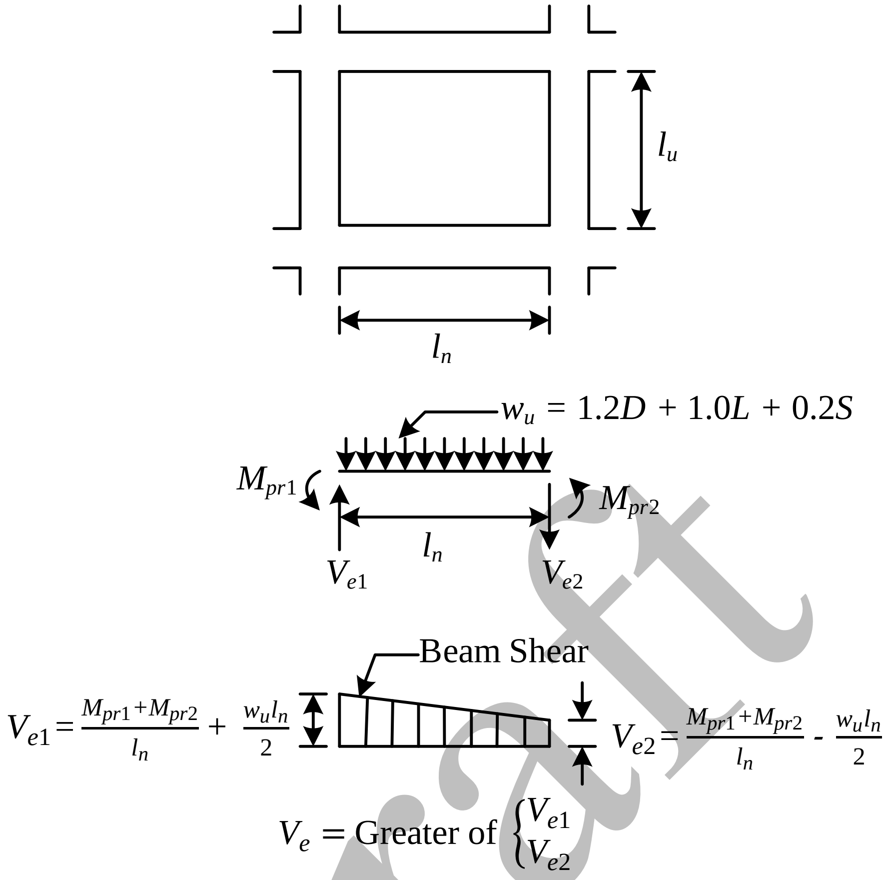

Fig. C3-1 The diagram of design beam shear imposed from the probable flexural strength of beam plastic hinges occurred at the column faces.

(adopted from ACI 318-14)

Wang et al. (2015, 2016) statistically investigate test results of beam specimens with longitudinal reinforcement using SD 420 and SD 690 steels manufactured from Taiwan, Japan, and U.S., and analytical results indicate that the flexural strength corresponding to lateral drift ratio of about 3.5% results in α_o *with an average of 1.25 and an upper bound of 1.31 (95% confidence level) for specimens using SD 420 steel. For specimens using SD 690 steel, the average and upper bound of* α_o *is 1.15 and 1.22, respectively. This design guideline suggests a minimum* α_o *of at least 1.20 for SD 690 steel.*

3.3.2 Nominal shear strength

Nominal shear strength of the beam is provided by concrete and transverse reinforcement

$$V_n = V_c + V_s \qquad (3\text{-}4)$$

Where V_n is nominal shear strength of the member, V_c is nominal shear strength provided by concrete, V_s is nominal shear strength provided by transverse reinforcement.

(1) Nominal shear strength from concrete

(a) Over the lengths equal to twice the beam depth on both sides of a section where flexural yielding is likely to occur.

$$V_c = 0 \qquad (3\text{-}5)$$

(b) Others

$$V_c = 0.17\sqrt{f_c'}b_w d \quad \text{(N, MPa, mm)} \qquad (3\text{-}6)$$

(2) Nominal shear strength from transverse reinforcement

$$V_s = \frac{A_v f_{yt} d}{s} \le 0.66\sqrt{f_c'}b_w d \quad \text{(N, MPa, mm, mm}^2\text{)} \qquad (3\text{-}7)$$

$$f_{yt} \le 600 \text{ MPa} \qquad (3\text{-}8)$$

Where A_v is area of transverse reinforcement within spacing s, f_{yt} is specified yield strength of transverse reinforcement, s is center-to-center spacing of transverse reinforcement.

[Commentary]

Nominal shear strength provided in this section primarily follows design *requirements of the "Design Specifications for Concrete Structures" (CPAMI, 2017). However, design strength of transverse reinforcement is different. Test results of cantilever beams subjected to cyclic loading (Wang et al., 2014) indicate the average recorded strain in transverse reinforcement is close to 600 MPa. As a result, this design guideline suggests that nominal shear strength provided by transverse reinforcement should be estimated by the smaller value of its specified yield strength and 600 MPa.*

3.4 Detailing

In addition to 3.4.1 and 3.4.2, reinforcement layouts of the beam should satisfy requirements of "Design Specifications for Concrete Structures".

3.4.1 Longitudinal Reinforcement

(1) At least two continuous bars at both top and bottom faces.

(2) Minimum flexural reinforcement ratio should be the greater of:

$$\rho_{s,\min} = \frac{0.25\sqrt{f_c'}}{f_y} \geq \frac{1.4}{f_y} \tag{3-9}$$

(3) Longitudinal reinforcement ratio for both top and bottom reinforcement should not exceed:

$$\rho_{s,\max} = \frac{f_c' + 9.8}{4 f_y} \leq 0.025 \tag{3-10}$$

(4) Lap splices should not be used in the locations of (a) through (c):

(a) Within the joints.

(b) Within a distance of twice the beam depth from the face of the joint.

(c) Within a distance of twice the beam depth on both sides of a critical section where flexural yielding is likely to occur.

3.4.2 Transverse Reinforcement

(1) Hoops should be provided in the following regions of the beam:

(a) Over a length equal to twice the beam depth from the face of the joint.

(b) Over a length equal to twice the beam depth on both sides of a section where flexural yielding is likely to occur.

(2) The first hoop should be located not more than 50 mm from the face of a supporting column. Spacing of the hoops should not exceed the least of (a) through (c).:

(a) $d/4$.

(b) Five times the diameter of the smallest primary flexural reinforcing bars.

(c) 150 mm.

(3) Where hoops are not required, stirrups with seismic hooks at both ends should be spaced at a distance not more than the lesser of $d/2$ and 300 mm.

(4) Spacing of transverse reinforcement enclosing the lap-spliced bars should not exceed the lesser of $d/4$ and 100 mm.

[Commentary]

3.4.1(2) and 3.4.1(3) intend to alleviate the steel congestion and limit shear stresses in beams. Requirements of 3.4.2(1), 3.4.2(2), and 3.4.2(3) are schematically illustrated in Fig. C3-2. Lap-spliced is not allowed in beam-column joints and in regions where flexural yielding is likely to occur.

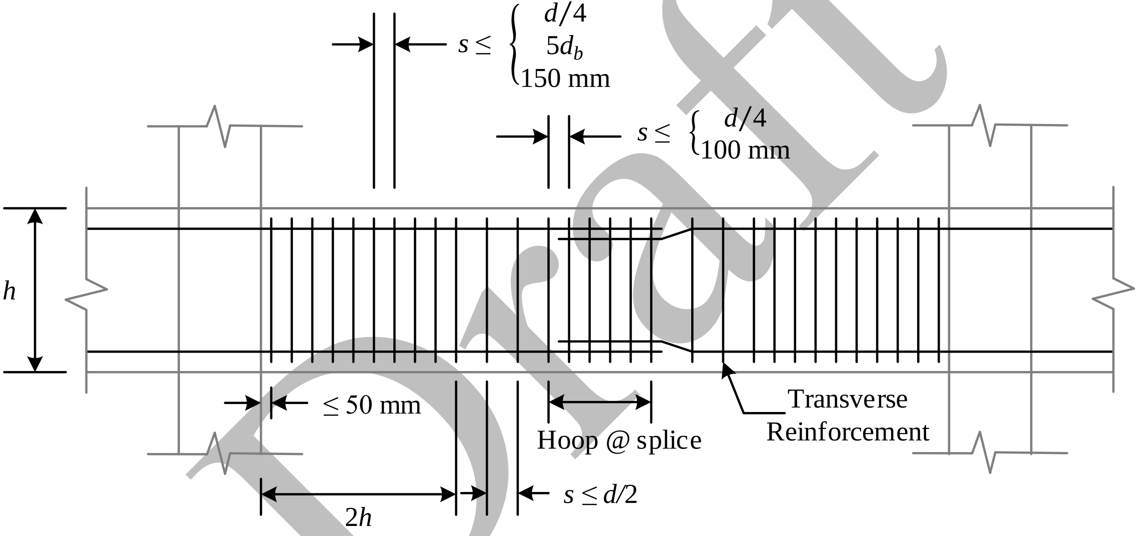

Fig. C3-2 Hoop and stirrup location and spacing requirements. (adopted from Moehle et al., 2008)

3.5 Crack Control

3.5.1 Scope

This chapter introduce relevant regulations of beam members in managing shear and flexural cracks when subjected to service load.

[Commentary]

In terms of shear crack control, when beam is subjected to service load (dead load and live load), beam section design shear stress shall not exceed the allowable shear stress recommended in this chapter, so as to make sure the maximum shear crack width is less than 0.4 mm. In terms of flexural crack control, this chapter follows the provisions of the ACI 318-14 (ACI, 2014), to control the spacing between longitudinal bars subjected to tensile stress. This ensure the maximum flexural crack width of beam members is less than 0.4~0.5 mm when sustaining service load.

ACI 318-14 does not give any relevant regulations regarding shear crack control method. Therefore in this chapter, we are mainly referring to the "Standard for Structural Calculation of Reinforced Concrete Structures," (AIJ, 2010). By limiting the allowable stress of concrete and reinforcement, we can achieve the purpose of maximum shear crack width control.

3.5.2 Shear Crack Control

When beam members are subjected to service load, the design shear stress of the section cannot be greater than the allowable shear stress v_{ser} obtained by Eq. (3-11):

$$v_{ser} = \frac{0.35 \times 0.33\sqrt{f_c'}}{1.5} + \frac{A_v f_s}{b_w s} \quad \text{(MPa, mm, mm}^2\text{)} \qquad (3\text{-}11)$$

Where A_v is the area of transverse reinforcement within spacing s, s is the center-to-center spacing of transverse reinforcement, f_s is the allowable stress of transverse reinforcement which does not exceed $0.15f_{yt}$, b_w is the web width,

f_c' is the specified compressive strength of concrete (MPa)

[Commentary]

The calculation of design shearing stress may refer to the suggestions given by ACI 318 standard, $v=V/(b_w d)$. The calculation of beam members' cross section shear stress when subjected to service loads, does not consider the service load amplification factor. The first term of Eq. (3-11) refers to the allowable shear stress of concrete. According to experimental results, shear span of the member is related to the effective depth ratio. For general beam members, it is recommending to calculate using Eq. (3-11) (Fang-Qing Lin, 2014). From Eq. (3-11), we can find that by increasing transverse reinforcement ratio, allowable shear stress of beam members is also raised. In order to effectively control the use of shear cracks under service load, when the span ratio of beam members increase, the demand for transverse reinforcement will also increase. The demand will be more than the amount of transverse reinforcement in accordance to seismic design. On contrary, if the span ratio of the beam member is below 12, transverse reinforcement demand is designed according to seismic design provisions, it is therefore need not to check the provisions regarding this chapter. (Fang-Qing Lin, 2014).

3.5.3 Flexural Crack Control

Spacing of longitudinal bars subjected to tensile stress shall not exceed the smallest of equations below:

$$s \leq 380\frac{280}{f_s} - 2.5\,C_c \qquad \text{(mm, MPa)} \tag{3-12}$$

$$s \leq \frac{300 \cdot 280}{f_s} \quad \text{(mm, MPa)} \tag{3-13}$$

Where f_s is the longitudinal reinforcement stress under service load which should not be taken less than $0.4f_y$, C_c is the clear cover from concrete surface to steel bars. If the outermost tensioned surface only consists of a single longitudinal reinforcement, s will be referring to the outermost surface width.

[Commentary]

According to present stage experimental results, beam flexural crack control may refer to the ACI 318-14 standard (ACI, 2014) set. However, when subjected to service load, beam steel bars' stress is low. In this condition, the distance between flexural cracks calculated according to Eqs. (3-12) and (3-13) seems to be too small, which would affect the conservativeness of the calculated crack width. Therefore, f_s should not be taken less than $0.4f_y$ in this equation, so as to decide the distance s of longitudinal reinforcement under greatest allowable stress value. (Shao-Chan Chen, 2015)

Reference

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[3.15] Yung-Chih Wang, Li-yan Hong, 2014, "Flexural Shear Behavior of New RC Beams," National Center for Research on Earthquake Engineering (NCREE), NCREE-14-010, Taipei, Taiwan. (in Chinese)

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[3.17] Yung-Chih Wang, Po-Yen Chen, 2015, "The Effect of High Strength Stirrup Layout on Seismic Behavior of New RC Beams," National Center for Research on Earthquake Engineering (NCREE), NCREE-15-004, Taipei, Taiwan. (in Chinese)

Chapter 4 Columns

Yu-Chen Ou, Wen-Cheng Shen, and Yi-An Li

4.1 Scope

Provisions in this chapter apply to columns subjected to flexure, axial, and shear loads and with a factored axial force P_u larger than $0.10A_g f_c'$, where A_g and f_c' are the gross cross-sectional area of a column and specified concrete compressive strength, respectively.

The dimension of the cross section of a column should satisfy the following requirements:

(1) The least dimension of the cross section, measured on a straight line passing through the geometric centroid, should not be less than 300 mm.

(2) The ratio of the least dimension to the dimension perpendicular to it should not be less than 0.4.

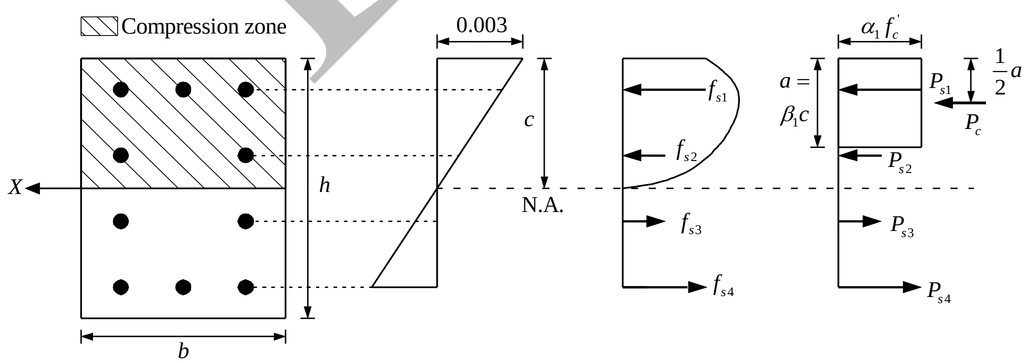
Design aspects not covered by this chapter should conform to "Design Specifications for Concrete Structures" and "Construction Specifications of RC Structures" for Reinforced Concrete Structures issued by Construction and Planning Agency, Ministry of the Interiors of Taiwan

4.2 Axial and Flexural Strengths

4.2.1 Ultimate State and Equivalent Stress Block for Concrete Compression Zone

When calculating the axial and flexural strengths, the ultimate strain of the extreme compression fiber of concrete should be assumed equal to 0.003. The tensile strength of concrete should be assumed equal to zero. The stress of steel reinforcement should be calculated by multiplying the strain of the reinforcement by E_s of steel reinforcement. When the strain is larger than the yield strain, the stress of steel reinforcement should be assumed equal to f_y and is independent of the strain. A concrete stress of $\alpha_1 f_c'$ should be assumed uniformly distributed over an equivalent compression zone of the cross section. The equivalent compression zone is bounded by the edges of the cross section and a line parallel to the neutral axis of the section and at a distance of $a = \beta_1 c$ to the extreme compressive fiber of the section, as shown in Fig. C4-1. α_1 and β_1 should be calculated by Eq. (3-1) and (3-2), respectively.

[Commentary]



(a) *Cross section* (b) *strain distribution* (c) *stress distribution* (d) *equivalent stress block of concrete and resultant forces*

Figure C4-1 The ultimate condition of the cross section.

4.2.2 Axial Strength

The axial strength at zero eccentricity P_0 should be calculated based on Eq. (4-1), where α_1 should be calculated by Eq. (3-1).

$$P_0 = \alpha_1 f_c^{'} \left(A_g - A_{st} \right) + f_y A_{st}, \quad f_y \le 600\, MPa \quad (\mathrm{N, MPa, mm^2}) \tag{4-1}$$

The maximum nominal axial strength of the section with spiral transverse reinforcement $P_{n,max}$ should be calculated based on Eq. (4-2). The maximum nominal axial strength of the section with rectilinear tie reinforcement $P_{n,max}$ should be calculated based on Eq. (4-3).

$$P_{n,max} = 0.85 P_0 \tag{4-2}$$

$$P_{n,max} = 0.8 P_0 \tag{4-3}$$

where A_{st} is the total cross-sectional area of longitudinal reinforcement; and f_y is the specified yield strength of longitudinal reinforcement.

[Commentary]

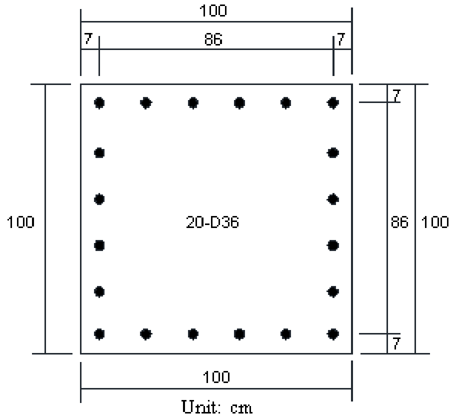
The axial strength at zero eccentricity P_0 is recommended by ITG-4.3R-07 (ACI, 2007). The stress of the equivalent stress block at the ultimate condition of the section is changed from $0.85f_c^{'}$ of the current code to $\alpha_1 f_c^{'}$. It is noted that because the ultimate compressive strain of concrete is 0.003. the ultimate compressive strain of longitudinal reinforcement is limited to 0.003 The corresponding compressive stress is 600 MPa. Reinforcement in tension is not subjected to this limit.

4.2.3 Strength under Combined Axial and Flexure Loads

The calculation of the strength under combined axial and flexure loads should conform to 4.2.1 to 4.2.2 and be based on equilibrium, “plane sections remain plane,” and compatibility.

[Commentary]

A computer code, NewRC-PM (Ou and Tsai 2016), has been developed with this manual for calculating the interaction diagram for axial loads and uniaxial moments or the interaction surface for axial loads and biaxial moments. An analysis example using NewRC-PM for a high-strength reinforced concrete section subjected to an axial load and a uniaxial moment, and an axial load and a biaxial moment is shown below. The section dimension is 1000×1000 mm. The concrete compressive strength f_c' *is 70 MPa and the yield strength of longitudinal reinforcement* f_y *is 690 MPa. D36 bars are used for the longitudinal reinforcement and are uniformly distributed along the perimeter of the section. Loading 1 contains an axial load* P_u = *1,500 tf and a uniaxial moment* M_{ux} = *600 tf-m. Loading 2 contains an axial load* P_u = *1,500 tf and a biaxial moment with* M_{ux} = *600 tf-m and* M_{uy} = *600 tf-m. The strength reduction factor conforms to the design code (CPAMI, 2017). Figure C4-2 shows the section design generated by NewRC-PM. Figure C4-3 shows the relationship between the interaction diagram for uniaxial bending and loading 1. It can be seen that loading 1 falls within the interaction diagram, which means the section is safe under loading 1. Figures C4-4 and C4-5 show the relationship between the interaction surface for biaxial bending and loading 2. The loading falls outside the interaction surface, which means the section is unsafe under loading 2.*



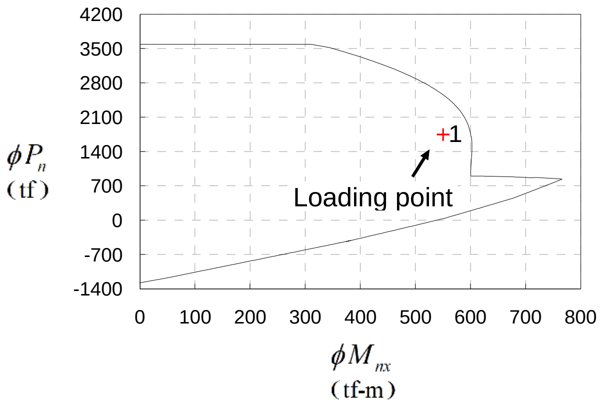


Figure C4-2 Section design.

Figure C4-3 Relationship between the interaction diagram for uniaxial bending and loading 1.

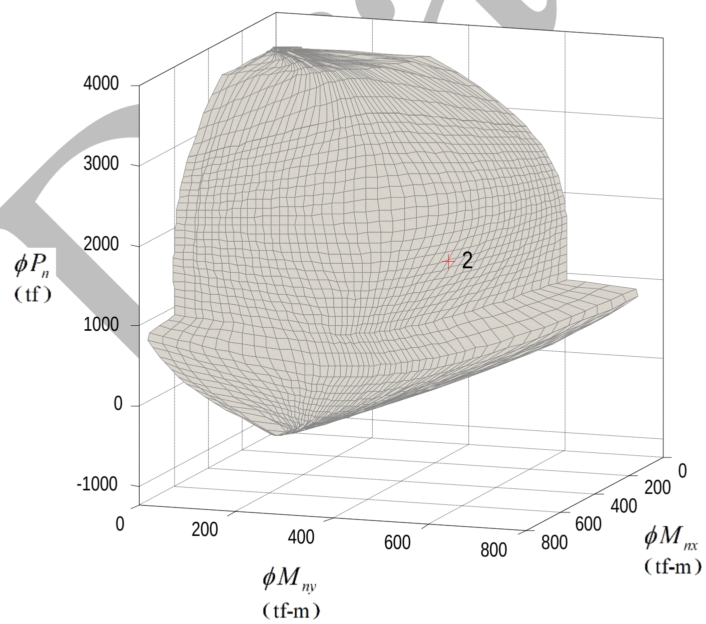


Figure C4-4 Relationship between the interaction surface for biaxial bending and loading 2.

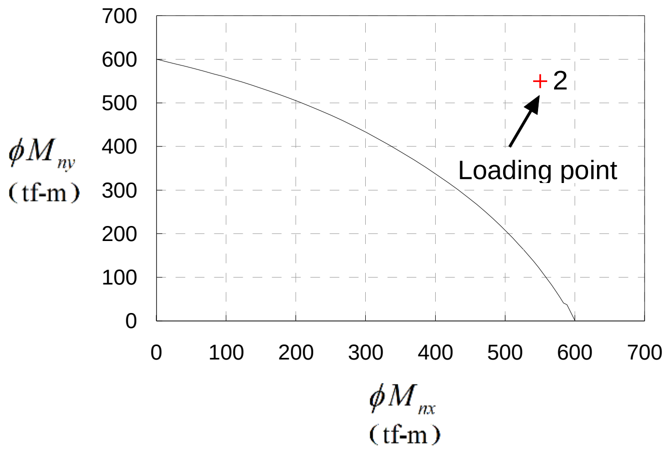


Figure C4-5 Relationship between the moment strength contour corresponding to P_u = 1750 tf and loading 2.

4.3 Shear Strength

4.3.1 Requirements for Shear Strength

The design shear V_e should be calculated considering the maximum probable moment strengths at each end of the column associated with the range of factored axial loads acting on the column. The maximum probable strength of the column should be calculated based on 4.3.2. The design shear needs not exceed the shear corresponding to the development of the maximum probable moment strengths at the ends of the beams framing into the same joints as the column. It should not be less than the factored shear acting on the column calculated by structural analysis. When V_e over the length l_0 exceeds half the required shear design and the factored axial compressive load including the earthquake effect is less than $0.05A_g f_c'$, V_c should be assumed equal to zero in determining the amount of transverse reinforcement over l_0. V_c is the nominal shear strength of concrete and l_0 is the length over which special transverse reinforcement must be provided.

l_0 should not be less than:

(1) The depth of the column at the end of the column or at the section where flexural yielding is likely to occur.

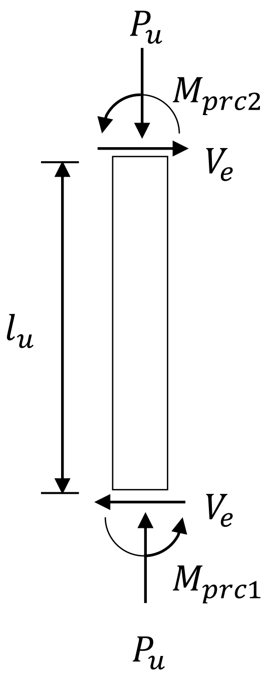
(2) One-sixth of the clear span of the column.

(3) 450 mm.

Transverse reinforcement required in 4.4 should be provided over l_0 from each end of the column and on both sides of any section where flexural yielding is likely to occur as a result of inelastic lateral displacements of the frame.

[Commentary]

Due to strong-column-weak-beam design, plastic hinges are expected to occur at the beam ends first. When the maximum probable moment strengths have been developed at the ends of the beams framing into a joint, the corresponding moment at the ends of the columns framing into the same joint can be calculated by multiplying the elastic moments acting at the ends of the columns from structural analysis by a ratio of the sum of the maximum probable moment strengths to the sum of the elastic moments from structural analysis at the ends of the beams. Both the moments at the top and bottom ends of a column M_{prc1} *and* M_{prc2} *can be calculated in this way. The maximum probable moment strength of a beam should be calculated according to 4.3.1. The design factored shear is* $V_e = (M_{prc1} + M_{prc2}) / l_u$*, where* l_u *is the clear height of the column, as shown in Fig. C4-6. The definition of* l_0 *is the same as the current design code (CPAMI, 2017).*



Note:

1. M_{prc} should be calculated based on 4.3.2 and should be the maximum probable strength considering all the factored axial forces.
2. V_e needs not exceed the shear corresponding to the development of the maximum probable moment strengths at the ends of the beams but should not be less than the factored shear calculated by structural analysis.

Figure C4-6 Calculation of the factored shear of a column.

4.3.2 Maximum Probable Moment Strength

The maximum probable moment strength of a column for shear design should be calculated by moment-curvature analysis considering the effects of concrete confinement, strain hardening of steel, and the actual strengths of materials higher than the specified strengths or by Eq. (4-4), where Ω_M should be calculated by Eq. (4-5). In Eq. (4-4), M_n is the maximum nominal moment strength associated with the range of factored axial loads acting on the column. In Eq. (4-5), P_u is the factored axial load and is positive for compression and negative for tension, and P_b is the nominal axial strength at the balanced strain condition.

$$M_{prc} = \Omega_M M_n \tag{4-4}$$

$$\Omega_M = \begin{cases} 1.31\left(P_u/P_0 - 0.13 \right) + 1.3 \geq 1.3, & P_u/P_b > 1 \\ 1.3, & P_u/P_b \leq 1 \end{cases} \tag{4-5}$$

[Commentary]

*According to the current design code, the maximum probable moment strength of a column is based on a stress of 1.25*f_y *in longitudinal reinforcement. When a column is subjected to a large axial compression (e.g. for columns in the lower stories of a high-rise building), the ultimate condition of the column tends to be compression controlled. In other words, the strain in the longitudinal tension reinforcement will be less than the yield strain. This means increasing stress to 1.25*f_y *in longitudinal tension reinforcement has no effect on the moment strength. The comparison between experimental and analytical results have shown that the* M_{prc} *calculated based on the current code significantly underestimates the test results (Ou and Tsai 2016). It is suggested in this manual to calculate the* M_{prc} *using moment-curvature analysis or Eqs. (4-4) and (4-5).*

The over-strength coefficient Ω_M *of Eq. (4-4) was obtained from comparing the test data of columns and analytical results (Ou and Tsai, 2016; Ou and Tsai 2018). Equation (4-5) shows the relationship between the axial force ratio* P_u/P_0 *and the over-strength coefficient* Ω_M*. For non-compression-controlled sections,* Ω_M *is set as 1.3, which is the same as the Seismic Bridge Design Specifications of Taiwan (MOTC, 2009). For compression-controlled sections,* Ω_M *is also set as 1.3 when* P_u/P_0 *is equal to or less than 0.13. When* P_u/P_0 *is larger than 0.13,* Ω_M *increases from 1.3 linearly with* P_u/P_0 *(Ou and Tsai, 2016; Ou and Tsai 2018). This reflects the fact that over strength has a strong positive correlation with the size of the compression zone of the section.*

4.3.3 Limits of Material Strengths

The value of $\sqrt{f_c^{'}}$ used in the calculation of shear strength provided by concrete should not be larger than $\sqrt{100\text{ MPa}}$. The steel stress used in the calculation of shear strength provided by shear reinforcement should not be larger than 600 MPa.

[Commentary]

Due to the lack of test data with concrete strength $f_c^{'}$ higher than 100 MPa and the fact that V_c calculated by equations in 4.3.5 tends to be less conservative with increasing $f_c^{'}$, the $\sqrt{f_c^{'}}$ used in the calculation of nominal shear strength is limited to $\sqrt{100\ MPa}$.

The test data of high-strength reinforced concrete columns (Liang 2015) have shown transverse reinforcement may not yield when the maximum shear strength of the column is reached. The test data also have shown that the stress in the transverse reinforcement increases with increasing amount of transverse reinforcement. When the minimum transverse reinforcement as required by 4.3.6 is provided, shear strength equations provided in 4.3.4 to 4.3.6 with a stress limit of 600 MPa in transverse reinforcement can provide predictions that are reasonably conservative as compared with the test data. Therefore, it is required in this manual that the stress in transverse reinforcement used in the shear strength calculation should not exceed 600 MPa.

4.3.4 Nominal Shear Strength

The nominal shear strength of a column V_n should be calculated using Eq. (4-6), where V_c is the shear strength provided by concrete and should be calculated by equations in 4.3.5 and V_s is the shear strength provided by transverse reinforcement and should be calculated by equations in 4.3.6.

$$V_n = V_c + V_s \tag{4-6}$$

[Commentary]

Same as the current design code, the nominal shear strength of a column consists of shear strength provided by concrete and that provided by transverse reinforcement.

4.3.5 Shear Strength Provided by Concrete

Shear strength provided by concrete V_c should be calculated by Eqs. (4-7) to (4-8) unless a more detailed calculation is made using Eqs. (4-9) to (4-12).

For columns subjected to shear and flexure only,

$$V_c = 0.17\sqrt{f_c^{'}}b_w d \qquad \text{(N, MPa, mm)} \tag{4-7}$$

where b_w is the width of the section and d is distance from the centroid of longitudinal tension reinforcement to the extreme compression fiber of the section.

For columns subjected to shear, flexure and axial compression

$$V_c = 0.17\left(1 + \frac{N_u}{14A_g}\right)\sqrt{f_c^{'}}b_w d \qquad \text{(N, MPa, mm)} \tag{4-8}$$

where N_u is the factored axial load acting together with V_u and is positive for compression and negative for tension.

The detailed equations for the nominal shear strength provided by concrete are

$$V_c = \left(0.16\sqrt{f_c^{'}} + 17\rho_w \frac{V_u d}{M_m} \right) b_w d \qquad \text{(N, MPa, mm)} \tag{4-9}$$

where ρ_w is longitudinal tension reinforcement ratio, which is defined as the area of longitudinal tension reinforcement divided by the effective sectional area; V_u is the factored shear of the section; and M_m is the factored moment modified to consider the effect of axial compression. M_m should be calculated by Eq. (4-10). When M_m is negative, V_c should be calculated by Eq. (4-11).

$$M_m = M_u - N_u \left(\frac{4h - d}{8} \right) \tag{4-10}$$

where M_u is the factored moment acting on the section; and h is the overall depth of the section.

V_c in Eq. (4-9) should not be larger than Eq. (4-11).

$$V_c = 0.29\alpha\sqrt{f_c^{'}} b_w d \sqrt{1 + \frac{2N_u}{\alpha\sqrt{f_c^{'}} b_w d}} \qquad \text{(N, MPa, mm)} \tag{4-11}$$

where α is a reduction factor to consider the effect of axial load and should be calculated by Eq. (4-12).

$$\alpha = \left(1 - 0.85\sqrt{\frac{N_u}{A_g f_c^{'}}} \right), \quad 0 \le \frac{N_u}{A_g f_c^{'}} \le 0.6 \tag{4-12}$$

[Commentary]

Eqs. (4-7) to (4-10) are the same as those used in the current design code (CPAMI, 2017). Eq. (4-11) differs from that used in the current design code and can better consider the effect of concrete strength and axial compression on the shear strength provided by concrete (Ou and Kurniawan, 2015a and 2015b). Eq.

(4-12) can consider the behavior of concrete in which the tensile strength in one principal direction can be reduced by axial compression acting along the other principal direction. Eq. (4-12) was derived from test data of high-strength concrete subjected to biaxial tension and compression forces. The use of α in Eq. (4-11) can reflect the decrease effect of axial compression on the diagonal cracking strength of concrete. This together with the behavior in which the diagonal cracking stress of concrete decreases with increasing axial compression leads to the behavior in which the increase of concrete shear strength approaches an upper limit at high axial compression as shown in Fig. C4-7.

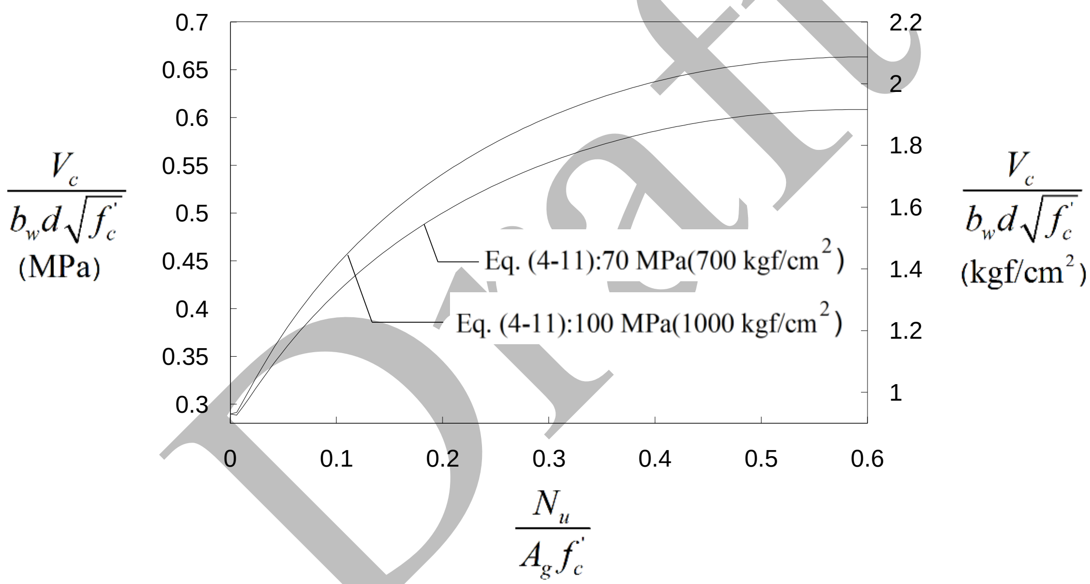


Figure C4-7 Behavior of concrete shear strength with increasing axial compression.

4.3.6 Shear Strength Provided by Transverse Reinforcement

The shear strength provided by transverse reinforcement V_s should be calculated based on Eq. (4-13).

$$V_s = \frac{A_v f_{yt} d}{s} \qquad (4\text{-}13)$$

where A_v is the total cross-sectional area of transverse reinforcement within spacing s; f_{yt} is the specified yield strength of transverse reinforcement; and s is the center-to-center spacing of transverse reinforcement. The V_s calculated by Eq. (4.13) should not be larger than the value calculated by Eq. (4-14).

$$V_{s,max} = 0.66\sqrt{f_c'} b_w d \qquad \text{(N, MPa, mm)} \qquad (4\text{-}14)$$

The minimum amount of transverse reinforcement $A_{v,min}$ should be

$$A_{v,min} = \frac{0.38 V_c s}{f_{yt} d}\beta \qquad (\text{mm}^2\text{, MPa, N, mm}) \qquad (4\text{-}15)$$

where β is a coefficient to reflect the effect of axial compression and is calculated by the following equation.

$$\beta = \frac{3N_u}{A_g f_c'} + 0.4, \quad 1.0 \le \beta \le 1.3 \qquad (4\text{-}16)$$

[Commentary]

Before formation of diagonal cracking due to shear, the stress in transverse reinforcement is typically small and transverse reinforcement takes little shear forces. When diagonal cracking occurs, part of internal forces originally taken by concrete will be redistributed to transverse reinforcement. Failure may occur if the amount of transverse reinforcement is not sufficient to allow for this redistribution of internal forces. To prevent failure right after diagonal cracking, a minimum amount of transverse reinforcement should be provided. The minimum amount of transverse reinforcement required by the current code (CPAMI, 2017) is independent of axial compression. Test results have shown that diagonal cracking strength increases with increasing axial compression. Therefore, the minimum amount of transverse reinforcement needs to be *increased with increasing axial compression to prevent sudden shear failure at diagonal cracking (Liang, 2015). The V_c and β in Eq. (4-15) are used to consider the effect of axial compression on the required minimum amount of transverse reinforcement.*

4.4 Confinement Requirement

The amount of transverse reinforcement for required confinement of rectilinear hoops shall be in accordance with (1) and (2).

(1) If the axial load applied on the column is less than or equal to $0.3A_g f_c'$ and the specified compressive strength of concrete is less than 70 MPa, the amount of rectilinear hoop reinforcement shall not be less than required by Eq. (4-17) and (4-18).

(2) If the axial load applied on the column is greater than $0.3A_g f_c'$ or the specified compressive strength of concrete is greater than 70 MPa, the amount of rectilinear hoop reinforcement shall not be less than required by Eq. (4-17), (4-18) and (4-19).

$$\frac{A_{sh}}{sb_c} = 0.3\frac{f_c'}{f_{yt}}\left(\frac{A_g}{A_{ch}} - 1\right) \tag{4-17}$$

$$\frac{A_{sh}}{sb_c} = 0.09\frac{f_c'}{f_{yt}} \tag{4-18}$$

$$\frac{A_{sh}}{sb_c} = 0.2k_f k_n \frac{P_u}{f_{yt}A_{ch}} \tag{4-19}$$

where A_{sh} is the total cross-sectional area of transverse reinforcement, including crossties, within spacing s and perpendicular to dimension b_c; b_c is the cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area A_{sh}; A_{ch} is the cross-sectional area of a member measured to the outside edges of transverse reinforcement; k_f is the concrete strength factor; k_n is the confinement effectiveness factor; P_u is the factored axial force; to be taken as positive for compression and negative for tension. The upper limit of the specified yielding strength of transverse reinforcement is less than or equal to 800 MPa.

In Eq. (4-19), the concrete strength factor k_f and confinement effectiveness factor k_n are calculated according to Eq. (4-20) and Eq. (4-21).

$$k_f = \frac{f_c'}{175} + 0.6 \geq 1.0 \qquad \text{(MPa)} \tag{4-20}$$

$$k_n = \frac{n_l}{n_l - 2} \tag{4-21}$$

where n_l is the number of longitudinal bars or bar bundles around the perimeter of a column core with rectilinear hoops that are laterally supported by the corner of hoops or by seismic hooks. In Eq. (4-21), the confinement effectiveness factor k_n term is depending on the number of rectilinear hoops that are laterally supported by the corner of crossties with seismic hooks or hoop. This design guideline liberalized the specification for crossties, and the crossties with seismic hook and 90-degree hook in another end shall be alternated end for end along the longitudinal reinforcement and around the perimeter of the cross section. However, the 90-degree hook is not included in the confinement effectiveness factor k_n when calculating n_l. Fig. C4-9 (a) shows the example of calculating the value of n_l where n_l is equal to 10.

Spacing of transverse reinforcement shall not exceed the smallest of (1) through (3):

(1) One-fourth of the minimum column dimension

(2) Five times the diameter of the smallest longitudinal bar

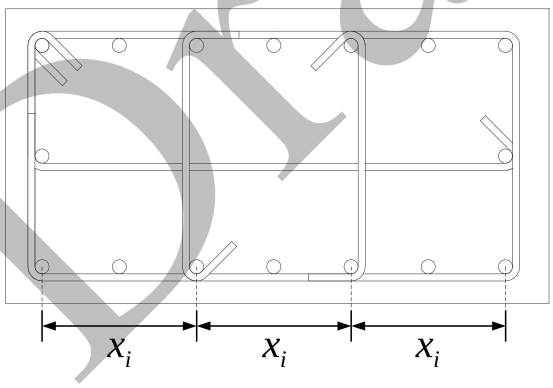
(3) s_0, as calculated by $s_0 = 100 + \left(\frac{350 - h_x}{3}\right)$ (mm)

where s_0 shall not exceed 150 *mm* and need not be taken less than 100 *mm*, h_x is the maximum center-to-center spacing of longitudinal bars laterally supported by corners of crossties or hoop legs around the perimeter of the column. The example of determining the value of h_x is shown in Fig. C4-8. As $P_u > 0.3A_g f_c'$ or $f_c' > 70$ MPa in columns with rectilinear hoops, the value of h_x shall not exceed 200 *mm*. On the contrary, $P_u \leq 0.3A_g f_c'$ and $f_c' \leq 70$ MPa, the value of h_x shall not exceed 350 *mm*, as shown in Fig. C4-9.

Other relevant provisions on rectilinear hoops are the same as in the current specifications.

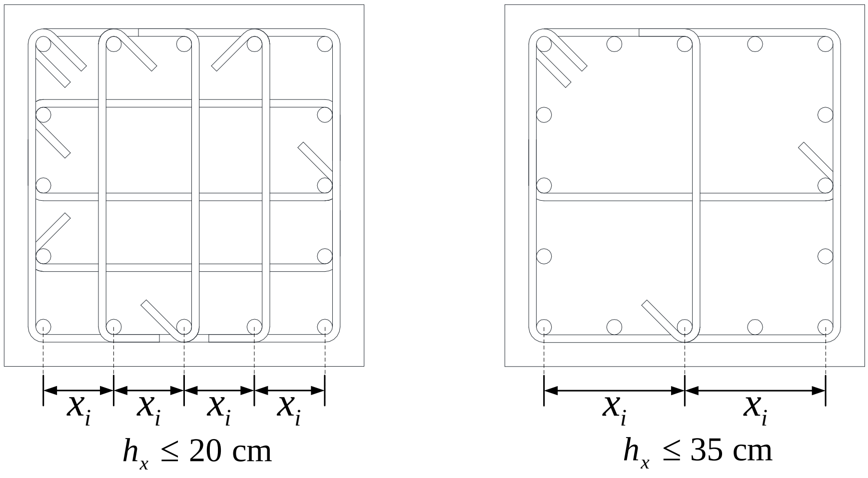
[Commentary]

Because the confinement effectiveness would be reduced by crossties with 90-degree hooks under high axial load, ACI 318-14 specified the improved confinement provided by having corners of hoops or seismic hooks supporting all longitudinal bars or bar bundles around the perimeter of a column core is important to achieve the intended performance. The requirement for confinement in ACI 318-14 is really strict and could be easily achieved by pre-assembling steel cages. However, in Taiwan, the construction method of on-site assembling steel cages is often adopted. Under the tight arrangement of longitudinal steel, the horizontal spacing between longitudinal steel is very small. The use of seismic hooks may increase the difficulty of construction. In *order to solve this problem, Hwang et al. (2013) suggested that the confinement effectiveness could be improved by using the crossties with seismic hook and 90-degree hook in another end being alternated end for end along the longitudinal reinforcement and around the perimeter of the cross section and the decreasing number of longitudinal bars or bar bundles around the perimeter of a column core with rectilinear hoops that are laterally supported by the corner of hoops or by seismic hooks. Based on test observations (Hwang et al. 2013), the deformation capacity of columns under high axial load could be sustained after peak lateral load. Therefore, this design guideline liberalized the specification for crossties in ACI 318-14, and the crossties with seismic hook and 90-degree hook in another end shall be alternated end for end along the longitudinal reinforcement and around the perimeter of the cross section. In addition, the amount of transverse reinforcement shall be increased to improve the confinement effectiveness.*



$$h_x = \max\{x_i\}$$

Fig. C4-8 Example of determining the value of h_x.



(a) $P_u > 0.3A_g f_c'$ *or* $f_c' > 70$ MPa

(b) $P_u \leq 0.3A_g f_c'$ *and* $f_c' \leq 70$ MPa

Fig. C4-9 Spacing h_x of longitudinal bars laterally supported by the corner of a crosstie or hoop leg in different condition.

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Chapter 5 Beam-Column Joints

Hung-Jen Lee

5.1 Scope

This chapter shall apply to beam-column joints of special moment frames forming part of the seismic-force resisting system, provided that conditions (1) through (5) are satisfied:

(1) Concrete shall be normalweight and have a design compressive strength f_c' not exceeding 100 MPa

(2) Specified yield strength of longitudinal reinforcement, f_y, shall not exceed 690 MPa

(3) Specified yield strength of transverse reinforcement, f_{yt}, shall not exceed 800 MPa

(4) Depth h_c of the joint, parallel to the shear direction, shall not be less than one-half of depth h_b of the beam framing into the joint and generating the joint shear

(5) Projection of the beam width shall not beyond the width of the supporting column

Other than the special provisions addressed in this chapter, the design shall be conformed to the ACI 318-14.

5.2 Shear strength

5.2.1 Joint Shear Force

The design shear force shall be computed on a horizontal plane at the mid-height of the joint by considering the forces acting on the joint. The stress $\alpha_o f_y$ in the flexural tensile reinforcement of the beams framing into the joint shall be in accordance with Table 5-1.

Table 5-1 α_o factor for reinforcement grade.

Reinforcement Grade	SD 490W	SD 550W	SD 690
Overstrength factor	α_o = 1.25	α_o = 1.25	α_o = 1.20

[Commentary]

Development of inelastic rotations at the faces of joints of reinforced concrete frames is associated with strains in the flexural reinforcement well in excess of the yield strain. Therefore, the design forces generated by the flexural reinforcement at the beam-column interfaces should be calculated using the stress of $\alpha_o f_y$. The overstrength factor α_o is intended to account for the bar actual yield stress and strain hardening when plastic hinging occurs. The Grade 490W, 550W, and 690 reinforcements are developed for earthquake-resistant concrete structures, and their ascending parts of the stress-strain relations are very similar to that of Grade 420, except for the offset of yield strength. Bar test data shows the ratio of actual-to-specified yield strength decreases as the reinforcement grade increases, thus the use of α_o = 1.25 for Grade 690 reinforcement could be too conservative.

The design shear force should be determined for the intended beam yielding mechanism adjacent to the joint. As shown in Fig. C5-1, the horizontal joint shear is generated by the flexural tension and compression resultants consistent with the beam probable moment strength M_{pr} *and column shear* V_{col}*. Although the location of column inflection point may be not always at the center of the column clear height in each story, it is acceptable to estimate the column shear* V_{col} *by assuming the column inflection point is at the mid-height in each column. (Moehle, 2015)*

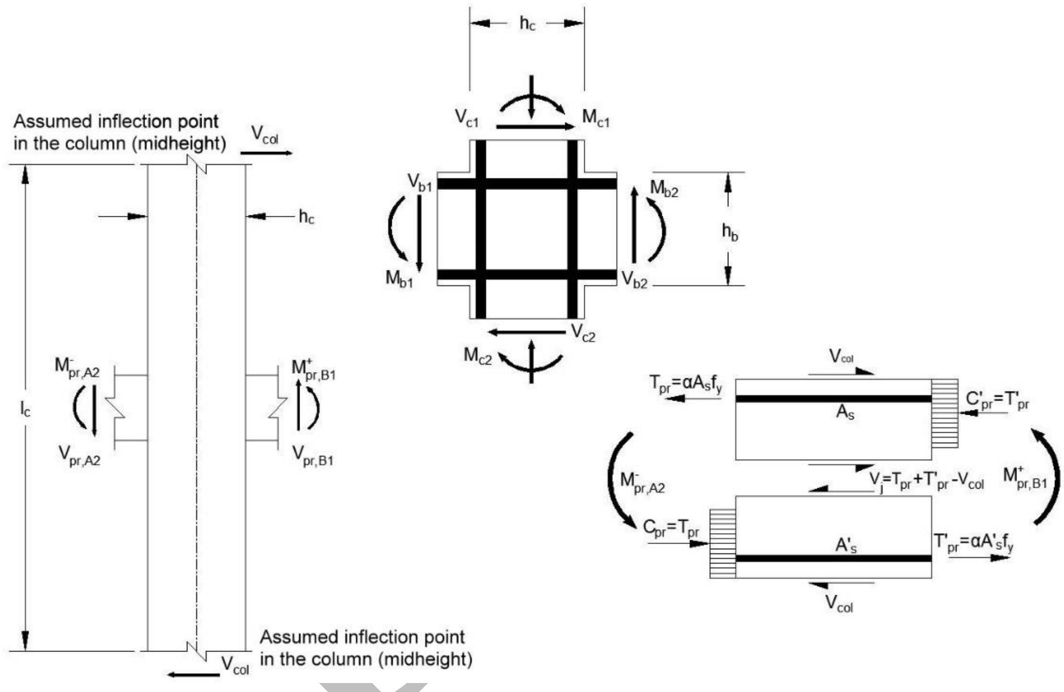

Fig. C5-1 Determination of joint horizontal shear for an interior joint of special moment frame. (Moehle, 2015)

5.2.2 Nominal Joint Shear Strength

The design shear force V_j of beam-column joints shall not exceed the design shear strength ϕV_n, where ϕ=0.85 and V_n, the nominal shear strength, is

$$V_n = 0.083\gamma\sqrt{f_c^{'}}A_j \qquad \text{(N, MPa, mm}^2\text{)} \qquad (5\text{-}1)$$

The constant γ for Eq. (5-1) is given in Table 5-2. The beam-column joints are classified according to whether the column and beam are continuous or discontinuous in the direction of joint shear being considered, and whether transverse beams confine the joint.

Table 5-2 Values of γ for beam-column joints.

Column Continuity[(a)]	Beam Continuity[(b)] in the direction of V_j	Confined by Transverse Beams[(c)]	Values of γ
Continuous	Continuous	Confined	20
		Not Confined	15
	Other	Confined	15
		Not Confined	12
Discontinuous	Continuous	Confined	15
		Not Confined	12
	Other	Confined	12
		Not Confined	8

Note : [(a)] A column is considered to be continuous through a beam-column joint if the column extends above the joint at least one overall column depth h_c. Extensions of columns for this purpose shall include continuation of longitudinal and transverse reinforcement from the column below the joint. The column longitudinal bars terminated in the extension shall be capable of developing bar yield strength at the joint-extension face.

[(b)] A beam in the direction of loading is considered to be continuous through a beam-column joint if the beam extends at least one overall beam depth h_b beyond the joint face with continuation of longitudinal and transverse reinforcement from the beam in the opposite side of the joint. The beam longitudinal bars terminated in the extension shall be capable of developing bar yield strength at the joint-extension face.

[(c)] A joint is considered to be confined by two transverse beams if the width of each transverse beam is at least three-quarters of the width of the joint face. Extensions of beams satisfying above condition [b] are considered adequate for confining that joint face.

[d] When a joint is considered to be confined by two transverse beams if the width of each transverse beam is between 1/2 to 3/4 of the width of the joint face, the γ value is calculated by linear interpolation.

Effective cross-sectional area within a joint, A_j, shall be calculated from joint depth times effective joint width. Joint depth shall be the overall depth of the column, h_c, in the direction of loading. Effective joint width shall be the overall width of the column, b_{col}, except where a beam frames into a wider column, effective joint width shall be calculated by

$$b_j = b + x_1 + x_2 \le b_{col} \tag{5-2}$$

where x_1 and x_2 are the effective width beyond the beam web to the column side on each side of the beam web. The width of x_1 or x_2 shall not be taken greater than $h_c/4$. (Hwang et al., 2014)

[Commentary]

Database investigation and experimental evidences confirm the applicability of existing ACI nominal shear strength for beam-column joints with high-strength concrete and high-strength steel reinforcement (Lee and Hwang, 2013; Lee et al., 2014; Lee et al., 2015; Lee and Chang, 2017). The shear force acting on the joint is essentially resisted by a diagonal concrete strut in the joint. A joint having a depth less than half the beam depth, or the anchorage length of beam longitudinal bars not exceeding 0.75h_c, would require a steep diagonal compression strut across the joint, which may be less effective in resisting joint shear and not applicable with this chapter.

For narrow beam-wide column connections, database investigation shows the effective joint width specified in ACI 318-14 may be too conservative for eccentric beam-column joints but unconservative for concentric joints. Hwang et al. (2014) evaluated a test database of beam-to-column connections and proposed the effective joint width of Eq. (5-2) and Fig. C5-2, which can improve the strength predictions of Eq. (5-1) for concentric or eccentric narrow beam-to-column connections.

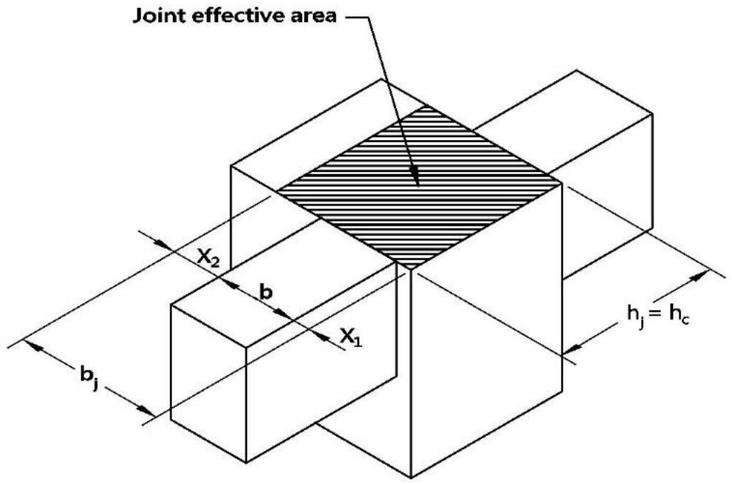

Fig. C5-2 Effective joint area.

The eight levels of shear strength provided by 5.2.2 are based on the recommendations of ACI-ASCE 352R (ACI, 2002). The values of γ given in Table 5-2 are now defined in terms of continuities of the column and beam in the direction of joint shear being considered, and confinement of the joint provided by transverse beams. Prior cyclic loading tests of joints evaluated in UT Austin (Meinheit and Jirsa, 1981; Zhang and Jirsa, 1982) indicated that transverse beams improved the joint capacity if two transverse beams are unloaded and present on both sides of the joint. The shear resistance provided by the diagonal concrete compressive strut increases linearly with the ratio of the transverse beam width (b_{tr}) to the column depth (h_c). The strut strength could be multiplied *by an empirical formula of 0.85+0.3 (b_{tr}/h_c) for $b_{tr}/h_c \geq 0.5$. (Zhang and Jirsa, 1982). The AIJ standards (AIJ, 2010) considered a joint to be confined by two transverse beams whenever $b_{tr}/h_c \geq 0.5$, and the reduction factor for a joint without confinement from transverse beams is 0.85. On the other hand, ACI-ASCE 352R and ACI 318-14 (ACI, 2014) conservatively considered a joint to be confined by two transverse beams whenever $b_{tr}/h_c \geq 0.75$, and the reduction factor for a joint without confinement from transverse beams is 0.75 or 0.80 (15/20=0.75, 12/15=0.80). Notably, the reduction factors given by AIJ or ACI 318 are consistent with the empirical formula of 0.85+0.3(b_{tr}/h_c), except the thresholds of b_{tr}/h_c is taken at 0.50 or 0.75, respectively.*

Reinforced concrete moment frames are commonly used seismic-force-resisting systems for building structures for many years. In common practice, typical beam width could cover 75% of the joint face to meet the threshold of $b_{tr}/h_c \geq 0.75$ per ACI 318. Unfortunately, it may not be satisfied for tall buildings with large and wide columns and would make the joint proportion difficult. In view of this, Lee et al (2016) tested high-strength reinforced concrete beam-column connections with transverse beams and slabs. The test results indicated that the strength of the joint with $b_{tr}/h_c = 0.67$ is only somewhat inferior to the joint with $b_{tr}/h_c = 0.75$. The researchers proposed that linear interpolations between the values of γ given in Table 5-2 can be applied to the joint confined by two transverse beams with b_{tr}/h_c ranges between 0.50 and 0.75, as shown in Fig. C5-3.

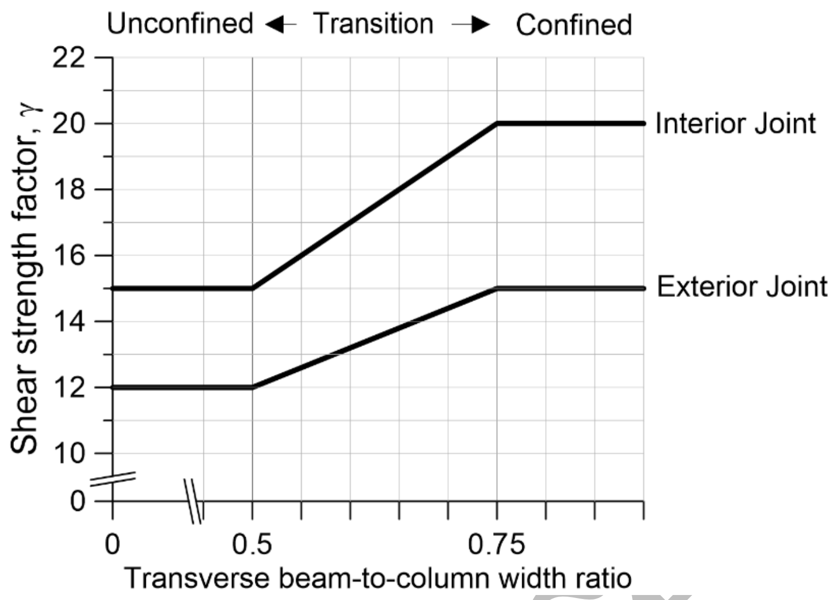

Fig. C5-3 Interpolations for values of γ for varied transverse beam-to-column width ratios.

5.3 Joint Transverse Reinforcement

Joint transverse reinforcement shall satisfy Section 4.4 in each direction, except for beams frame into two opposite faces of a joint and where each beam width is at least 3/4 of the column width, transverse reinforcement perpendicular to those two covered faces shall be permitted to be reduced to not less than half that required in Section 4.4, and the vertical spacing shall not exceed the smaller of $5d_b$ and 150 mm within the overall depth of the beam.

Where beam negative moment reinforcement is provided by headed deformed bars that terminate in the joint, the column shall extend above the top of the joint a distance at least the depth of the joint. Alternatively, the beam reinforcement shall be enclosed by additional vertical joint reinforcement providing equivalent confinement to the top face of the joint.

[Commentary]

Because tests of joints (Meinheit and Jirsa, 1981) indicated that shear strength was not as sensitive to joint transverse reinforcement as implied by the conventional shear formulas for beams, ACI-ASCE 352R set the strength of the joint as a function of only the compressive strength of the concrete and requires a minimum amount of transverse reinforcement in the joint. The ACI code requires transverse reinforcement in a joint regardless of the magnitude of the calculated shear force. The amount of confining reinforcement may be reduced and the spacing may be increased if beams of adequate dimensions frame into all four sides of the joint (Meinheit and Jirsa, 1981; Ehsani and Wight, 1985; Durrani and Wight, 1985).

Following the development of high-strength concrete, Saqan and Kreger (1998) evaluated test results from 26 beam-column connections with concrete compressive strengths ranging from 42 to 107 MPa and indicated the amount of transverse reinforcement in the joint could be reduced for joints with high-strength concrete. Noguchi et al. (1998) also presented a Japanese database of 110 beam-column joints with concrete compressive strength over 60 MPa, and concluded that transverse reinforcement was marginally effective in increasing joint shear strength, and that the effect of transverse reinforcement on joint shear strength was not sensitive to concrete compressive strength. They also found that the effect of transverse reinforcement was slightly more significant for exterior joints than for interior joints. (ACI ITG-4.3R-07)

More recently, Lee and Hwang (2013) assembled a database of cyclic tests for 357 beam-column joints made with all grades of reinforcement and concrete. In this database investigation, all 357 reinforced concrete joint specimens are concentric beam-column joints without transverse beams. The evaluation of test *database by Lee and Hwang (2013) concluded the ACI design provisions yielded conservative estimates of strength for concrete compressive strengths up to 100 MPa, and the code limitation of* $f_{yt} \leq 690$ *MPa for confinement reinforcement is conservative and could be liberated.*

Joint transverse reinforcement is relatively important in exterior or corner joints without framing beams on the opposite faces of the joint. The amount of transverse reinforcement required in Section 4.4 is necessary for maintaining the exterior joint integrity and delaying the joint strength deterioration under large inelastic deformation reversals (Lee and Chang, 2017). Although experimental results showed that beam-column joints with low amounts of transverse reinforcement were able to attain shear strengths comparable with those of code-conforming joints, experimental results also show more transverse reinforcement could delay the deterioration of joint shear capacity and further increase the drift capacity corresponding to the joint shear failure.

5.4 Development Length Requirements for Beam-Column Joints

5.4.1 Minimum Column Dimension Parallel to the Beam Reinforcement Extends Through a Beam-Column Joint

Where longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall be at least the larger of (1) and (2).

(1) $\dfrac{\alpha_o f_y}{4\sqrt{f_c'}}$ (mm, MPa) (5-3)

(2) $20d_b$

where d_b is the diameter of the largest longitudinal beam bar extends through the joint.

[*Commentary*]

ACI-ASCE 352R and ACI 318-14 set a minimum joint depth of $20d_b$*. This* $20d_b$ *criterion is based on an evaluation (Zhu and Jirsa, 1983) of cyclic loading tests of 18 beam-column joints made with 28-35 MPa concrete and Grade 420 MPa reinforcement. The researchers concluded that a minimum column depth of* $20\text{-}22d_b$ *is appropriate to avoid excessive bar slip at an interstory drift of 3% for Grade 420 MPa reinforcement.*

More recent tests, however, indicate that the $20d_b$ *criterion will not completely prevent bond damage and excessive bar slip within the joint (Quintero-Febres and Wight, 2001; ACI-ASCE 352R). Relatively pinched hysteresis behavior can be observed in the beam-column joints with bond damage along the beam bars passing through the joint. Such damage in the joint core is unlikely to be easily repairable and therefore should be avoided in a design basis earthquake event.*

While the minimum column dimension requirement in ACI 318-14 is regardless of material properties, design criteria in the AIJ (2010) and NZS 3101 (2006) standards are relatively rational to set a minimum h_c/d_b *ratio as a function of the bar* f_y*, concrete compressive strength, reinforcement ratio, and column axial load. In view of that the* $20d_b$ *criterion cannot be simply extended for beam-column joints made with higher grade reinforcement, Lee et al. (2018) summarizes international existing design criteria and proposes Eq. (5-3) for the minimum joint depth. The applicability of Eq. (5-3) is assessed by evaluating the cyclic testing results of beam-column joints conducted in East Asian and Pacific Countries, where Grade 490, 590, and 690 reinforcement have been used for earthquake-resistant concrete structures. Beam-column joints that satisfy the* *proposed equation can demonstrate satisfactory hysteresis behavior at an interstory drift of 4%.*

5.4.2 Development Length Requirements for Beam Bars Terminated Within a Joint

The development length ℓ_{dh} of a beam longitudinal bar terminating in a standard hook within a joint shall be calculated by Eq. (5-4), and ℓ_{dh} shall be at least the greatest of 8d_b, 150 mm, and 3/4 of the column depth.

$$\ell_{dh} = \frac{f_y d_b}{5.4\sqrt{f_c^{'}}} \quad \text{(mm, MPa)} \tag{5-4}$$

For headed deformed bars satisfying following conditions (1) through (3), the development length ℓ_{dt} of a headed bar within a joint shall be calculated by Eq. (5.5), and ℓ_{dt} shall be at least the greatest of 8d_b , 150 mm, and 3/4 of the column depth.

$$\ell_{dh} = \frac{f_y d_b}{5.2\sqrt{f_c^{'}}} \quad \text{(mm, MPa)} \tag{5-5}$$

Conditions:

(1) Net bearing area of head A_{brg} shall be at least 4A_b;

(2) Value of $\sqrt{f_c^{'}}$ used to calculate development length shall not exceed $\sqrt{100\text{ MPa}}$;

(3) Clear spacing between bars shall be at least 2d_b.

[Commentary]

Eq. (5-4) is a combination of the provisions in Section 25.4.3 of ACI 318-14. Chapter 5 stipulates that the hook is to be embedded in confined concrete, the modification factors 0.7 (for concrete cover) and 0.8 (for transverse reinforcement) have been incorporated in Eq. (5-4). An increase in length is also factored into the equation to reflect of load reversals and probable overstrength in bar stress.

Eq. (5-5) is given by ACI 318-14 for development length of headed deformed bars in tension, which is conditionally used for bar f_y *not exceeding 420 MPa, value of* $\sqrt{f_c^{'}}$ *not exceeding* $\sqrt{70\,MPa}$ *, and clear spacing between bars not less than 3*d_b*. Section 5.4.2 modify the conditions according to several extensively database investigations (Kang et al., 2009; Lee and Hwang, 2013; Ou et al., 2017) and experimental verifications (Chiu et al., 2016; Lee and Chang, 2017).*

Fig. C5-4 shows example details for interior and exterior beam-column joints. Engineers should consider that the vertical column bars and horizontal beam bars from two directions must pass by each other with adequate spacing and cover dimension. To preclude anchorage failure and to promote the development of a diagonal compression strut in the joint for shear resistance, headed bars should satisfy the minimum anchorage length requirement of ℓ_{dt} *and extend through the joint, over 3/4 of the overall joint depth, to the far face of the confined core, even though the resulting anchorage length exceeds the* ℓ_{dt}*. (Lee and Chang, 2017)*

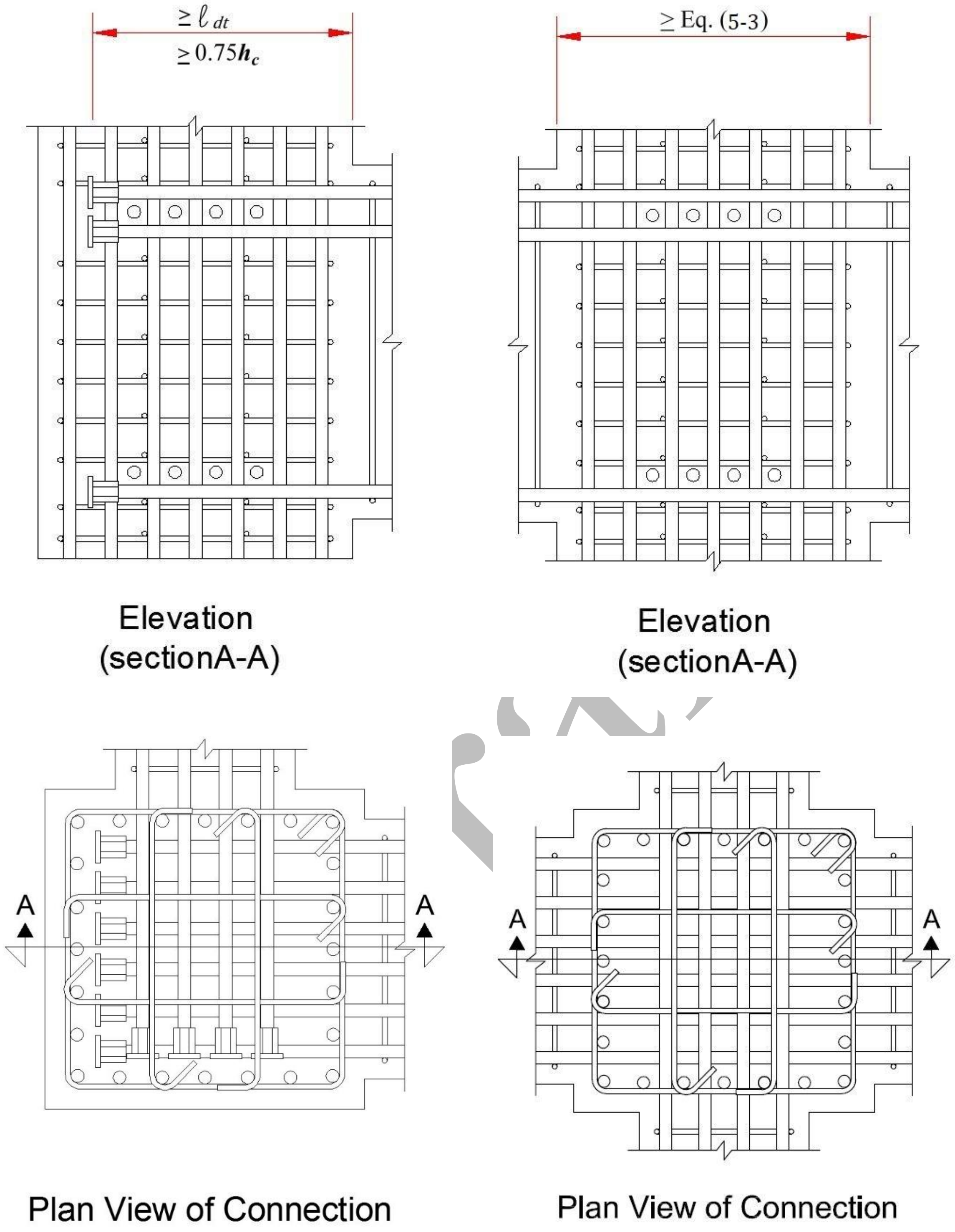

Fig. C5-4 Example details for beam-column joints.

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Chapter 6 Walls and Coupling Beams

Chung-Chan Hung, Ming-Yuan Cheng, Min-Lang Lin

6.1 Scope

Items not specified in this chapter shall conform to requirements of "Design Specifications for Concrete Structures" and "Construction Specifications of RC Structures". Specified material strengths shall be in accordance with (1) through (4):

(1) Normal weight concrete with a specified strength not exceeding 70 MPa.

(2) The specified yield strength for the longitudinal reinforcement in structural walls shall not exceed 790 MPa. The specified yield strength for the longitudinal and diagonal reinforcement in coupling beams shall not exceed 690 MPa.

(3) The design shear strength for transverse reinforcement shall not exceed 600 MPa.

(4) The design strength for confinement reinforcement shall not exceed 690 MPa.

6.2 Structural Wall

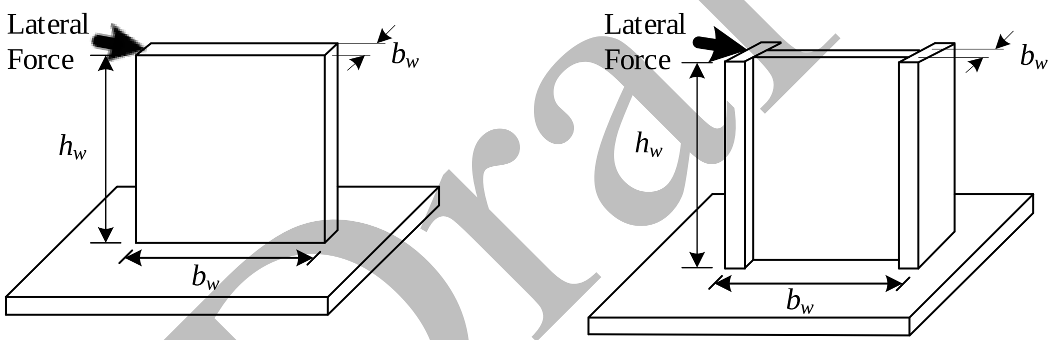
Design provisions of RC structural walls are classified by h_w/ℓ_w and ℓ_w/b_w of the wall, as shown in Table 6-1. In which, h_w is the wall height measured from the center of lateral loading to the critical section, ℓ_w is the wall length, and b_w and is the wall width.

Table 6-1 Design provisions for vertical wall segments.

Type	$(\ell_w/b_w)\leq 2.5$	$2.5<(\ell_w/b_w)\leq 6.0$	$(\ell_w/b_w)>6.0$
$(h_w/\ell_w)<2.0$	Wall (6.2)	Wall (6.2)	Wall (6.2)
$(h_w/\ell_w)\geq 2.0$	Wall piers required to satisfy column design requirements (Chapter 4)	Wall piers required to satisfy column design requirements (Chapter 4)	Wall (6.2)

[Commentary]

The wall geometry is schematically illustrated in Fig. C6-1.



(a) Rectangular cross section *(b) Barbell cross section*

Fig. C6-1 Wall Geometry.

6.2.1 Design Strength

Walls shall be designed for factored axial force, bending moment, and in-plane shear force.

6.2.1.1 Axial Force and Bending Moment

Interaction between axial and flexure loads shall be considered in the wall design to ensure demands from combined axial and flexure loads do not exceed the wall strength. Unless a more detailed analysis is performed, effective flange widths of flanged sections shall extend from the face of the web a distance equal to the lesser of (1) and (2):

(1) One-half the distance to an adjacent wall web.

(2) 25% of the total wall height.

Factored axial load shall not exceed $0.35A_g f_c'$, where A_g is the cross-sectional area of the wall web and the wall flange bounded by the effective flange width.

6.2.1.2 Shear

Shear demand shall not exceed the nominal shear strength. The nominal shear strength of the wall is determined in compliance with Eq. (6-1).

$$V_n = A_{cv}(\alpha_c\sqrt{f_c'} + \rho_t f_{yt}) \qquad \text{(N, mm, MPa)}$$

$$\alpha_c = \begin{cases} 0.25, & h_w/\ell_w \le 1.5 \\ 0.17, & h_w/\ell_w \ge 2.0 \\ \text{Interpolation}, & 1.5 < h_w/\ell_w < 2.0 \end{cases} \tag{6-1}$$

In which, A_{cv} is the gross area of the section bounded by web thickness and length of the wall, f_c' is the specified strength of normal-weight concrete, f_{yt} is the specified yield strength of the distributed horizontal web reinforcement, and ρ_t is the reinforcement ratio of the distributed horizontal web reinforcement. For horizontal wall segments, V_n shall not be taken greater than $0.67A_{cv}\sqrt{f_c'}\ (MPa)$.

[Commentary]

If linear-elastic analyses are used to determine axial and flexure strength demands of the wall, flexural and axial stiffness shall be reduced to consider possible development of cracks and loss of stiffness due to cyclic loading. Assessment of design recommendations from ACI 318-14 (ACI, 2014), CSA A23-3 (CSA, 2004), NZS 3101 (NZS, 2006) suggests that the moment of inertia of the wall could be taken as $0.7I_g$ *for regions outside the plastic hinge. Within the plastic hinge, the moment of inertial and area of the wall could be taken as and as* $0.5I_g$ *and* $0.5A_g$*, respectively, in the analysis. The height of plastic hinge can be assumed as the wall length,* ℓ_w*. Shear demands of the RC structural wall may be estimated from flexural strength of the wall. If the equivalent lateral load procedure is used in the analysis, the obtained base shear demand should be modified as* $V_u = \omega_v \alpha_o V_E$ *as suggested by Paulay and Priestley (1992). In which,* $\omega_v = 1.3 + \dfrac{n}{30} \leq 1.8$ *is dynamic modification coefficient to account for the high-mode effect on the base shear demand,* α_o *is overstrength factor of the longitudinal reinforcement as listed in Table 5-1,* V_E *is the base shear demand from structural analysis.*

The effective flange width of the flanged section shall extend from the face of the web a distance equal to the lesser of one-half the distance to an adjacent wall web and 25% of the total wall height, as schematically illustrated in Fig. C6-2. Based on limited test results, design strength of the distributed horizontal web reinforcement shall not exceed 790 MPa.

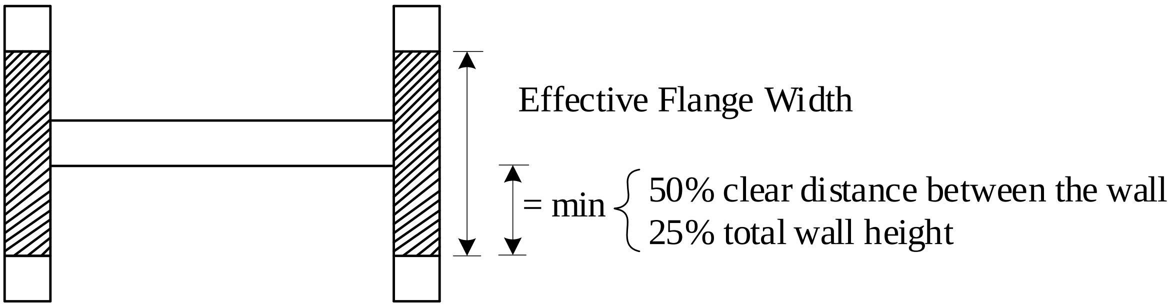


Fig. C6-2 Schematic Effective Flange Width.

Although not required by ACI 318-14 (ACI, 2014) and ASCE-7(ASCE, 2010), this design guideline refers to ASCE-41 (ASCE, 2013) and suggests the largest factored axial force not exceeding $0.35A_g f_c'$ *to avoid brittle failure mode.*

Equation (6-1) is based on provisions of ACI 318-14, however, this prediction model is controversial. Several strength prediction models have been proposed previously (Wood, 1990; Hwang et al., 2001; Gulec et al., 2008). Test results of RC low-rise walls using high-strength steels (Hung, et al., 2017) indicate that specimens designed with shear stress demand of $0.54\sqrt{f_c'}$ *(MPa) and subjected to lateral displacement reversals were able to sustain design forces up to 1.5% drift, meeting the requirements of ASCE-41. Considering that the specimen deformation capacity decreases as the shear stress demand increases (Cheng et al., 2016), this design guideline suggests* V_n *be less than* $0.67A_{cv}\sqrt{f_c'}\ (MPa)$ *to prevent premature brittle failure.*

6.2.2 Boundary Elements of Special Structural Walls

The need for special boundary elements at the edges of structural walls shall be evaluated under applicable load combinations that address the earthquake effect in accordance with (1) or (2). Where special boundary elements are required, 6.2.3 shall be satisfied.

(1) For walls with $h_w/\ell_w \geq 2.0$ designed to have a single critical section for flexure and axial loads and c exceeds the value from Eq. (6-2), special boundary element transverse reinforcement shall extend vertically above and below the critical section at least the greater of ℓ_w and $M_u/4V_u$.

$$c \geq \frac{\ell_w}{600(1.5\delta_w / h_w)} \tag{6-2}$$

where c corresponds to the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the direction of the design displacement δ_w. δ_w/h_w shall not be taken less than 0.005.

(2) Others

The wall shall have special boundary elements at boundaries and edges where the maximum extreme fiber compressive stress exceeds $0.2f_c'$. The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than $0.15f_c'$.

6.2.3 Details for Special Boundary Elements

Where special boundary elements are required, (1) though (10) shall be satisfied:

(1) The boundary element shall extend horizontally from the extreme compression fiber a distance at least the greater of $c - 0.1\ell_w$ or $c/2$.

(2) Width of the flexural compression zone over the horizontal distance calculated by 6.2.3(1) shall be at least $h_u/16$ to increase out-of-plane stability, where h_u is laterally unsupported height at extreme compression

fiber of the wall.

(3) In flanged sections, the boundary element shall include the effective flange width in compression and shall extend at least 300 mm into the web.

(4) Transverse reinforcement in special boundary elements shall comprise either single or overlapping spirals, circular hoops, rectilinear hoops with or without crossties.

(5) Bends of rectilinear hoops and crossties shall engage peripheral longitudinal reinforcing bars. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement and around the perimeter of the cross section.

(6) The diameter of transverse reinforcement in the special boundary element shall be at least D10 for D32 or smaller longitudinal reinforcement, and shall be at least D13 for longitudinal reinforcement with diameter greater than D32.

(7) Rectilinear ties in the special boundary element shall be arranged such that every corner and alternate longitudinal bars shall have lateral support provided by the corner of a crosstie or hoop. Around the perimeter of the special boundary element, spacing of longitudinal bars laterally supported by the corner of a crosstie or hoop shall not exceed the smaller of 350 mm and 2/3 of the boundary element thickness, and no unsupported longitudinal bars shall be father than 150 mm clear on each side from a laterally supported bar.

(8) Spacing of transverse reinforcement in special boundary elements shall

not exceed the smallest of (a) through (c):

(a) One-third of the least dimension of the boundary element;

(b) 5 times the diameter of the smallest longitudinal bar;

(c) s_o, as calculated by Eq. (6-3). s_o shall not exceed 150 mm and shall not be taken less than 100 mm.

$$s_o = 100 + \left(\frac{350 - h_x}{3}\right) \quad \text{(mm)} \tag{6-3}$$

(9) Amount of transverse reinforcement shall be in accordance with Table 6-2. In which, $f_{yt} \leq 800$ MPa and A_{be}, A_{ch} and b_c can be determined in accordance with Fig. C6-3.

Table 6-2 Transverse reinforcement for special boundary elements.

Transverse reinforcement	Applicable expressions	
$\frac{A_{sh}}{sb_c}$ $\frac{A_{sh}}{sb_c}$ for rectilinear hoop	Greater of	$0.3\left(\frac{A_{be}}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}}$ $0.09\frac{f_c'}{f_{yt}}$
ρ_s ρ_s for spiral or circular hoop	Greater of	$0.45\left(\frac{A_{be}}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}}$ $0.12\frac{f_c'}{f_{yt}}$

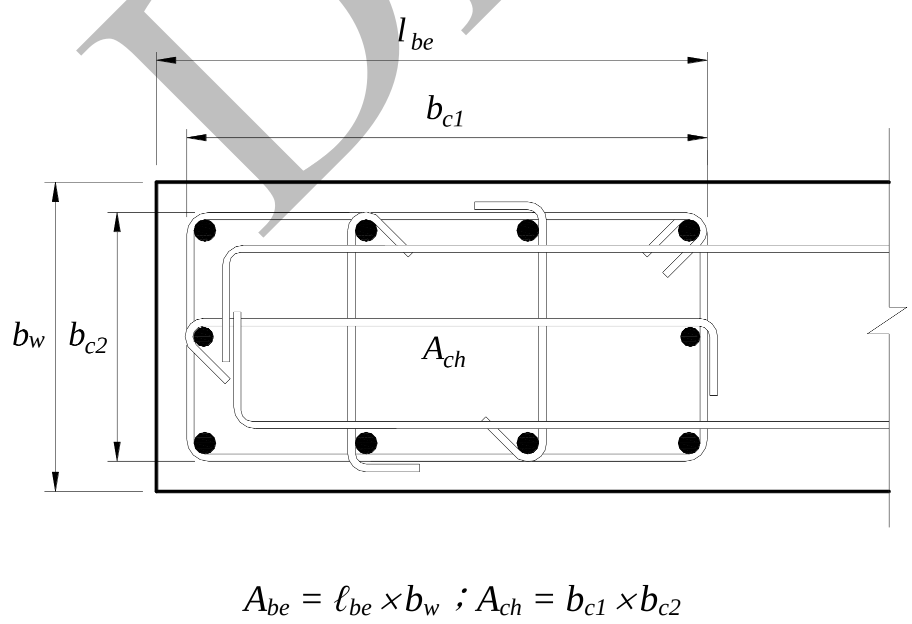
(10) Horizontal reinforcement in the wall web shall extend to within 150 mm of the end of the wall. Reinforcement shall be anchored to develop f_y within the confined core of the boundary element using standard hooks or heads. Where the confined boundary element has sufficient length to develop the horizontal web reinforcement, and $A_s f_y / s$ of the horizontal web

reinforcement does not exceed $A_s f_{yt}/s$ of the boundary element transverse reinforcement parallel to the horizontal web reinforcement, it shall be permitted to terminate the horizontal web reinforcement without a standard hook or head.

[Commentary]

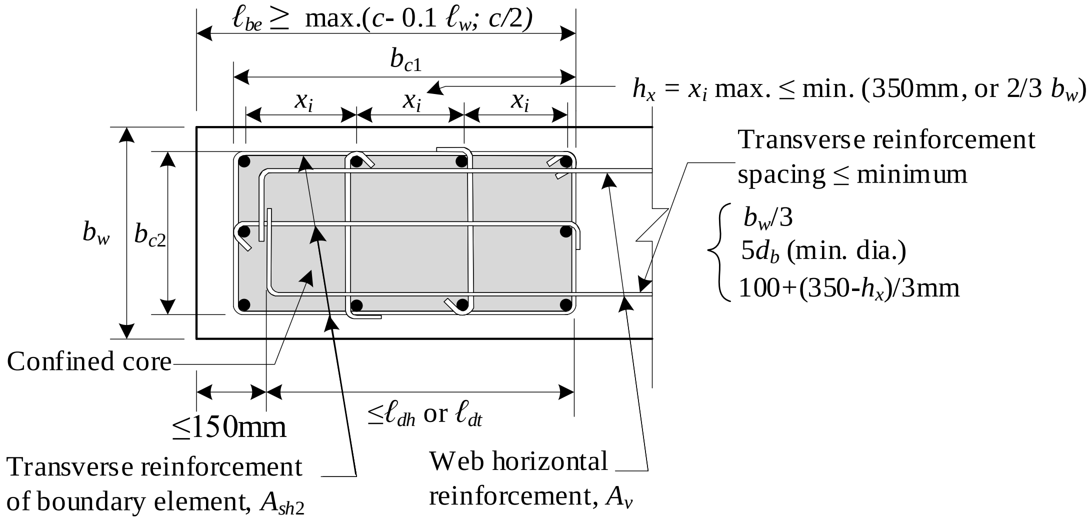
Special boundary elements at the edges of structural walls are needed to sustain design forces when the wall is subjected to large cyclic lateral loading such as earthquake. For low-rise walls, this design guideline suggests to determine the need of special boundary element using the stress method. The stress is to be calculated for the factored forces on the section assuming linear response of the gross concrete section. For walls with flanged sections, the effective flange width shall be in accordance with 6.2.1.

The terms A_{be}、A_{ch} and b_c used in Table 6-2 are schematically illustrated in Figs. C6-2 and C6-3. Other requirements are presented in Fig. C6-4.

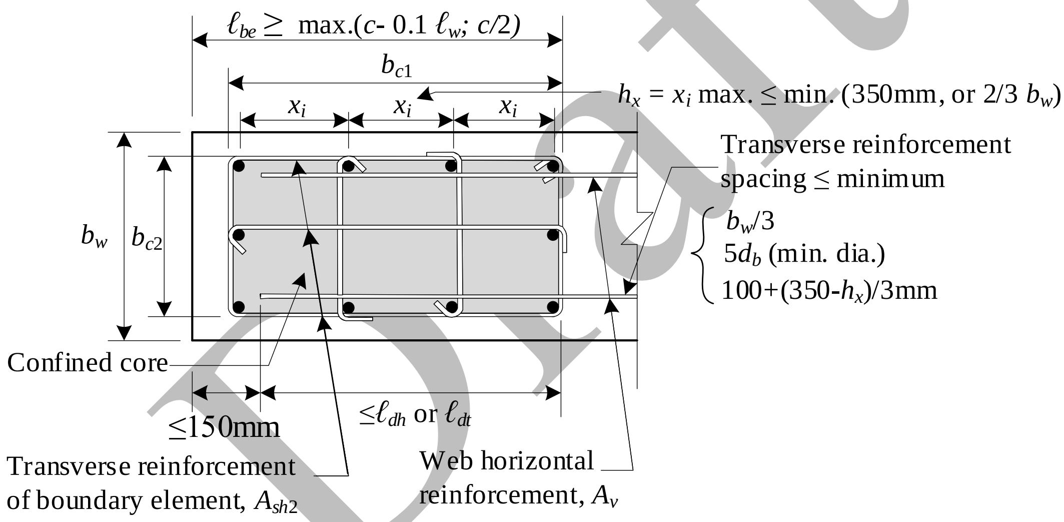


$$A_{be} = \ell_{be} \times b_w \text{ ；} A_{ch} = b_{c1} \times b_{c2}$$

Fig. C6-3 Geometry of wall special boundary elements.



(a) Option with standard hooks or heads.



(b) Option with straight reinforcement.

Fig. C6-4 Development of wall horizontal reinforcement.

6.2.4 Requirements for Non-Special Boundary Elements

Where special boundary elements are not required, (1) and (2) shall be satisfied:

(1) If the longitudinal reinforcement ratio at the wall boundary exceeds $2.8/f_y$ (MPa), boundary transverse reinforcement shall satisfy 6.2.3(1) through (10). The longitudinal spacing of transverse reinforcement at the wall boundary shall not exceed the lesser of 200 mm and $6.5d_b$, except the spacing shall not exceed the lesser of 150 mm and $5d_b$ within a distance equal to the greater of ℓ_w and $M_u/4V_u$ above and below critical sections where yielding of longitudinal reinforcement is likely to occur.

(2) Except where the shear demand in the plane of the wall is less than $0.083A_{cv}\sqrt{f_c^{'}}\ (MPa)$, horizontal reinforcement terminating at the edges of the structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

6.2.5 Other Detailing

(1) Where the critical section occurs at the wall base, the boundary element transverse reinforcement at the wall base shall extend into the support at least ℓ_d, where ℓ_d is determined by $\alpha_o f_y$ (see Table 5-1 for α_o) of longitudinal reinforcement with the largest diameter. Where the special boundary element terminates on a footing, mat, or pile cap, and the boundary element have an edge within 1/2 the footing (or other support) depth from an edge of the footing (or other support), special boundary element transverse reinforcement shall extend into the support at least ℓ_d, where ℓ_d is determined by $\alpha_o f_y$ (see Table 5-1 for α_o) of longitudinal reinforcement with the largest diameter. Otherwise, boundary element transverse reinforcement shall extent at least 300 mm into the footing,

mat, or pile cap.

(2) The distributed horizontal web reinforcement ratio ρ_t and vertical web reinforcement ratio ρ_ℓ for structural walls shall be at least 0.0025. At least two curtains of reinforcement shall be used in a wall if V_u is greater than $0.17 A_{cv}\sqrt{f_c'}\,(MPa)$, or $h_w/\ell_w \geq 2.0$. If does not exceed 2.0, reinforcement ratio ρ_ℓ shall be at least the reinforcement ratio ρ_t.

[Commentary]

Development of the wall longitudinal reinforcement, ℓ_d, shall be designed using $\alpha_o f_y$. Other detailing requirements are presented in Fig. C6-5.

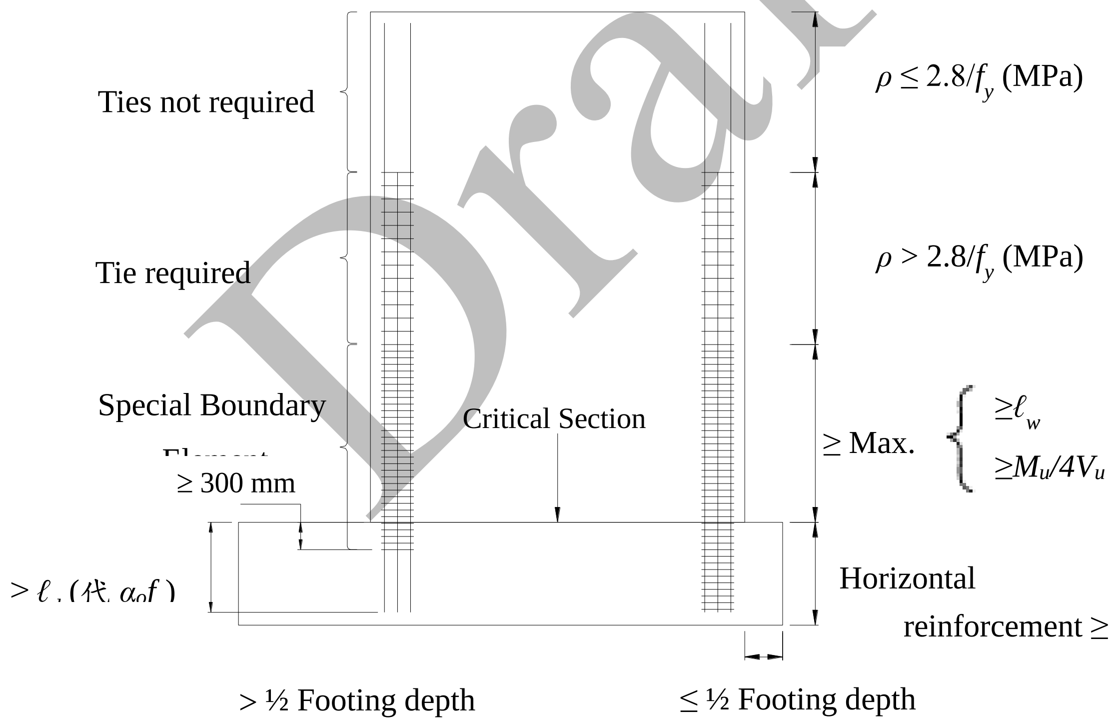


Fig. C6-5 Boundary element requirements for special walls.

Past research indicates that distributed vertical reinforcement in the web influences shear capacity of the low-rise walls (Barda et al., 1977; Wood, 1990).

As a result, ρ_ℓ is suggested to be at least ρ_t in low-rise walls.

6.3 Coupling Beams

Design provisions of RC coupling beams are classified by (ℓ_n/h), as shown in Table 6-3. In which, h is the height of beam, ℓ_n is the length of clear span of the beam.

Table 6-3 Design provisions for coupling beams.

aspect ratio	$(\ell_n/h)<2$	$2\leq(\ell_n/h)<4$	$(\ell_n/h)\geq4$
Design requirements	Sec. 6.3(2)	Sec. 6.3(3)	Sec. 6.3(1)

(1) Coupling beams with $\ell_n/h\geq4$ shall satisfy the requirements of flexural member (Chapter 3).

(2) Coupling beams with $\ell_n/h<2$ and with V_u greater than $0.34\sqrt{f_c'}A_{CW}$ (MPa) shall be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan, unless it can be shown that loss of stiffness and strength of the coupling beams will not impair the vertical load-carrying ability of the structure, the egress from the structure, or the integrity of nonstructural components and their connections to the structure.

(3) Coupling beams not governed by (1) or (2) shall be permitted to be reinforced either with two intersection groups of diagonally placed bars symmetrical about the midspan or according to the requirements of flexural member.

(4) Coupling beams reinforced with two inter-section groups of diagonally

placed bars symmetrical about the midspan shall satisfy the requirements of (a) and (b), the transverse reinforcement shall satisfy the requirements of (c) or (d).

(a) Shear strength V_n shall be calculated by

$$V_{\mathrm{n}} = 2A_{vd} f_y \sin\alpha \leq \quad 0.85\sqrt{f_c^{'}}A_{cw} \qquad \text{(N, mm, MPa)} \qquad (6\text{-}4)$$

In which, A_{vd} is the area of the diagonal bars, α is the angle between the diagonal bars and the longitudinal axis of the coupling beam, A_{cw} is the area of concrete section of coupling beam resisting shear.

(b) Each group of diagonal bars shall consist of a minimum of four bars provided in two or more layers. The diagonal bars shall be embedded into the wall at least $\alpha_o f_y$ times the development length in tension, α_o is listed in Table 5-1.

(c) Each group of diagonal bars shall be enclosed by rectilinear transverse reinforcement having out-to-out dimensions of at least $b_w/2$ in the direction parallel to b_w and $b_w/5$ along the other sides, where b_w is the web width of the coupling beam. The above transverse reinforcement shall meet the requirements of Section 4.4, but the equation (4-21) shall not be considered. For the purpose of calculating A_g, the concrete cover shall be assumed on all four sides of each group of diagonal bars. The transverse reinforcement shall continue through the intersection of the diagonal bars. Additional longitudinal and transverse reinforcement ratio shall be distributed around the beam perimeter with total area in each direction of at least 0.002 and spacing not exceeding 300 mm.

(d) The transverse reinforcement shall meet the requirements of Section

4.4, but the equation (4-21) shall not be considered. Longitudinal spacing of transverse reinforcement shall not exceed the lesser of 150 mm and 5 times diameter of diagonal bars. Spacing of crossties or legs of hoops both vertically and horizontally in the plane of the beam cross section shall not exceed 200 mm. Each crosstie and each hoop leg shall engage a longitudinal bar of equal or greater diameter.

[Commentary]

Two confinement options are described in this section. According to Section 6.3(4)(c), each diagonal element consists of a cage of longitudinal and transverse reinforcement, as shown in Fig. C6-6. Section 6.3(4)(d) describes a second option for confinement of the diagonals. This second option is to confine the entire beam cross section instead of confining the individual diagonals, as shown in Fig. C6-7. This option can considerably simplify field placement of hoops.

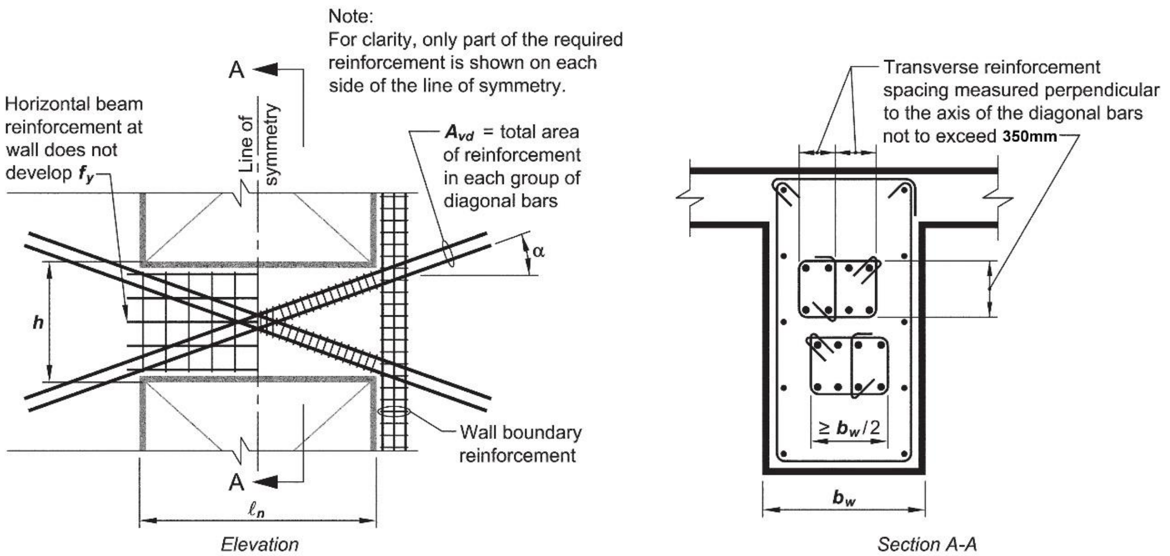


Fig. C6-6 Confinement of individual diagonals. (ACI, 2014)

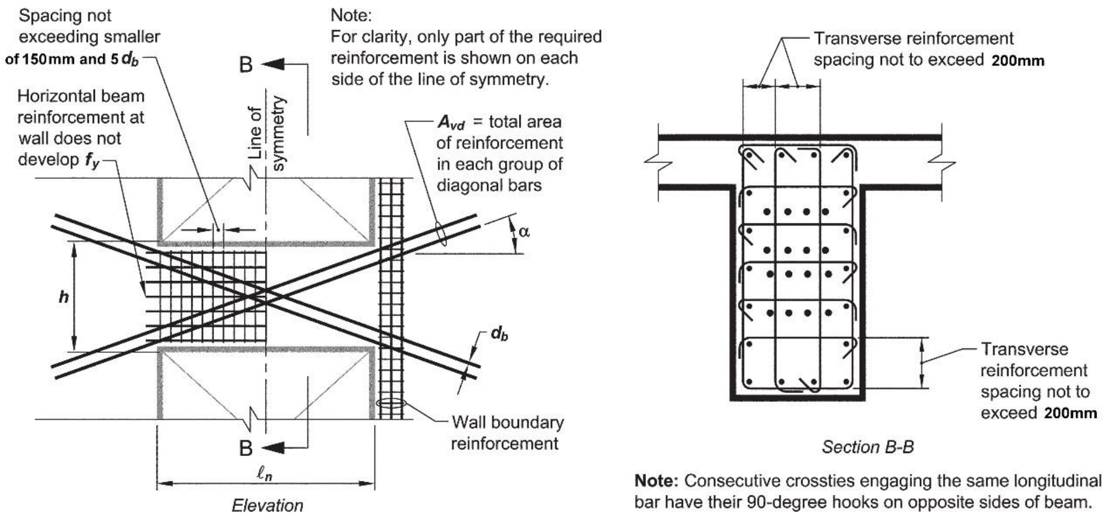


Fig. C6-7 Full confinement of diagonally reinforced concrete beam section.

(ACI, 2014)

Reference

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Chapter 7 Development and Splices of Reinforcement

Ker-Chun Lin, Yung-Chih Wang, Hung-Jen Lee, Kai-Ning Chi

7.1 Scope

The design requirements in this chapter applies to the development length, anchorage, and splice of the reinforced concrete members which shall satisfy (1) through (6):

(1) Concrete shall be normalweight and have a specified compressive strength f_c' not exceeding 100 MPa.

(2) The lightweight concrete shall not be used in this chapter.

(3) Specified yield strength of longitudinal reinforcement, f_y, shall not exceed 690 MPa

(4) The SD 690 steel reinforcement shall not be welded.

(5) The calculations of development and anchorage lengths do not require a strength reduction factor ϕ.

(6) The cut-off design of steel reinforcement shall conform to requirements of section 5.11 in "Design Specifications for Concrete Structures".

(7) Other than the special provisions addressed in this chapter, the design shall be conformed to "Design Specifications for Concrete Structures" and "Construction Specifications of RC Structures".

[Commentary]

The bond behavior between concrete and reinforcement is the most fundamental and important concept in relation to the mechanical aspects of RC.

The reinforcement and the concrete can form composite structures that can be loaded by the bonding force between two of them. The bond strength between the reinforcement and concrete arises from the interlocking behavior between the frictional resistance and binding force of the two materials. When a specified strength is provided by the reinforcement on a certain section of an RC member, the reinforcement must be configured with the straight extension, hooked, or mechanical anchorage. When the bond strength between the reinforcement and concrete is insufficient, concrete-bar slippage occurs, causing the thin protective layers of the concrete around the bar to split along the reinforcement.

In general, the straight development length of a reinforcing bar can be used to be the anchorage for developing its specified strength in concrete. When the straight development length of reinforcement cannot be applied owing to the limited size of the member, hooked or mechanical anchorage can be used to obtain the required development length of the bar.

However, the high-strength reinforcement in this design guideline contains large amounts of carbon, neither standard hooked nor welded connections can be used for the anchorage. Therefore, reinforcing bars with threaded surfaces (Fig. C7-1) replace deformed surfaces to generate convenient mechanical anchorages and splices with the uses of the threaded end-anchorage grout-filled device and threaded splice grout-filled sleeves (Fig. C7-2).

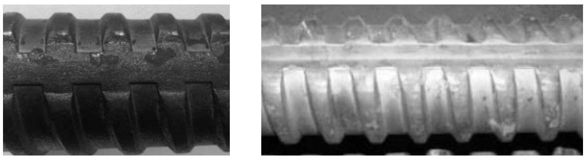

Fig. C7-1 Details of the threaded bar.

Fig. C7-2 Details of grout-filled end-anchorage device and splice sleeve.

National Center for Research on Earthquake Engineering (NCREE) launched a project called "The Taiwan New RC Project" to develop high-strength reinforced-concrete structural systems. This project sets the high-strength reinforcement with the specified yield strength 690 MPa longitudinal threaded bars for research. Additionally, the suggestions of design requirements in this guideline are followed the related studied results. Therefore, the specified yield strength of longitudinal reinforcement shall not exceed 690 MPa in this guideline.

According to the results of bonding tests of SD 690 threaded bars in NCREE (Lin et al, 2012, 2013, 2015), the bond performance of the threaded bars with diameters in the range of D25 (No.8) and D41 (No.14) can be predicted by using the formula in ACI 318-14 (ACI, 2014). However, the bond strengths of threaded bars with R_r *values that exceed 0.17 are all good enough to develop the required strengths when the strengths of concrete are not limited to 70 MPa, even the compressive strength of concrete is increased up to 100 MPa.*

Due to the lack of bond performance studies for high-strength deformed reinforcement (over than 490 MPa) and high-strength concrete (over than 70 MPa) in the world, the lower limit of the ratio of average rib height and average rib spacing of deformed reinforcement is 0.12 as recommended by the ACI 408-03 report (ACI, 2003).

7.2 Development of Reinforcement

7.2.1 Straight Development of Reinforcement

(1) Development of bars in tension

The development length for bars in tension, ℓ_d shall be calculated in accordance with Eq. (7-1) and (7-2), and ℓ_d shall not less than 300 mm. In Eq. (7-2), the splitting index, $(c_b+K_{tr})/d_b$ shall not exceed 2.5.

$$\ell_d = 0.9 \frac{f_y}{\sqrt{f_c^{'}}} \frac{\psi_t \psi_e \psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)} d_b \qquad \text{(mm, MPa)} \qquad (7\text{-}1)$$

$$1.0 \le \left(\frac{c_b + K_{tr}}{d_b}\right) \le 2.5 \qquad (7\text{-}2)$$

where f_y is the specified yield strength of reinforcement (MPa).

$f_c^{'}$ is the specified compression strength of concrete (MPa).

c_b is the smaller of the distance from the center of a bar to the nearest concrete edge or half center-to-center spacing of bars (mm).

d_b is the diameter of bars being developed (mm).

K_{tr} is the transverse reinforcement index (mm), and $K_{tr} = 40A_{tr}/sn$ in Eq. (7-2).

s is the spacing of transverse reinforcement (mm).

n is the number of bars being developed or lap spliced along the plane of splitting (mm).

A_{tr} is the total sectional area of all transverse reinforcement within spacing s being developed along the plane of splitting (mm).

ψ_t is the factor used to modify development length for casting location in tension.

ψ_e is the factor used to modify development length based on reinforcement coating.

ψ_s is the factor used to modify development length based on reinforcement size.

(2) Development of bars in compression

The development length for bars in compression, ℓ_{dc} shall be calculated in accordance with Eq. (7-3), and ℓ_{dc} shall not less than 200 mm.

$$\ell_{dc} = 0.24 \frac{f_y}{\sqrt{f_c^{'}}} d_b \geq 0.043 f_y d_b \qquad \text{(mm, MPa)} \tag{7-3}$$

(3) Development of bundled bars

Development length for individual bars within a bundle, in tension or compression, shall be that of the individual bar, increased 20 percent for a three-bar bundle, and 33 percent for a four-bar bundle.

[Commentary]

The confining effect of longitudinal bars is included in the calculation of the development length of reinforcement (Eq. (7-1)). In general, the confining effect is consisted of the c_b *(a factor that represents the side cover or the center-to-center spacing of the bars) and* K_{tr} *(a factor that represents amounts of transverse confinement). The bonding strength of the bar increases with the confinement. However, past researches and ACI 318-14 indicate* $(c_b+K_{tr})/d_b$ *is related to the bonding failure modes. When* $(c_b+K_{tr})/d_b$ *is less than 2.5, the splitting failure is likely to be occurred. For the value beyond 2.5, the pullout failure could be expected. Thus, an increase in cover or transverse reinforcement cannot increase the anchorage capacity. Therefore, the calculation of* $(c_b+K_{tr})/d_b$ *in this design guideline shall not exceed 2.5.*

According to NCREE research on the bonding behaviors of threaded bars, it was found the bonding failure mode still belonged to splitting failure mechanism for the splitting index $(c_b+K_{tr})/d_b$ *larger than 2.5. Therefore, it recommends that the upper limit, 2.5 of splitting index* $(c_b+K_{tr})/d_b$ *in the provision of ACI 318-14 for calculating the development length of steel reinforcement needs more test verifications.*

Until now, although the studies on bond of the high-strength reinforced *concrete are carried out, the modification factors of straight development lengths of bars in tension have not been discussed yet. Therefore, the modified factors as given in Table C7-1 remain the factors stipulated in ACI 318-14 code. The reduction factor based on excess reinforcement is not to be used in those cases where anchorage development for full f_y or seismic design is required.*

The development of bars in compression ℓ_{dc} can be reduced by multiplying the applicable factors in Table C7-2.

Table C7-1 The factors used for the development of bars in tension.

Conditions	*Factors*
factor used to modify length for casting location (ψ_t) (1) *Where horizontal reinforcement is placed such that more than 300 mm of fresh concrete is cast below the development length or splice.* (2) *For other situations.*	 *1.3* *1.0*
factor used to modify length based on reinforcement coating (ψ_e) (1) *For epoxy-coated bars with cover less than $3d_b$, or clear spacing less than $6d_b$.* (2) *For all other epoxy-coated bars.* (3) *For uncoated and zinc-coated (galvanized) reinforcement.*	 *1.5* *1.2* *1.0*
factor used to modify length based on reinforcement coating (ψ_e) (1) *For D19 and smaller bars.* (2) *For D22 and larger bars.*	 *0.8* *1.0*
factor used to modify length based on excess reinforcement. *(1) Development of rebar for f_y is specifically required.* *(2) The factor shall be permitted where reinforcement is in excess of that required by analysis.*	 *1.0* $\frac{A_s \text{ required}}{A_s \text{ provided}}$

Hint: the factors $\psi_t \psi_e$ shall not be greater than 1.7 for top reinforcement and epoxy-coated reinforcement.

Table C7-2 The factors used for the development of bars in compression.

Consideration	*Conditions*	*Factors*
excess reinforcement	*Reinforcement in excess of that required by analysis* *(1) Development of rebar for f_y is specifically required.* *(2) The factor shall be permitted where reinforcement is in excess of that required by analysis.*	*1.0* $\frac{A_s \text{ required}}{A_s \text{ provided}}$
spiral reinforcement	*Reinforcement enclosed within spiral reinforcement not less than 6 mm diameter and not more than 100 mm pitch.*	*0.75*
transverse reinforcement	*Reinforcement enclosed within D13 transverse reinforcement in conformance with 13.9.5 of "Design Specifications for Concrete Structures" and spaced at not more than 100 mm on center*	*0.75*

Wang et al. (2014 and 2016) studied the SD 690 beam bar cut-off designed according to the requirements of ACI 318-14 code to observe the seismic performance of the tested beams. The tested beams having SD 690-D25 rebars being cut-off and the specified concrete strength f_c' = 30 MPa are called as type-I beams, whereas the beams with cut-off rebars SD 690-D32 and the f_c' = 50 MPa are classified as type-II beams. Note that the cut-off points of bars for all specimens are controlled by the development length specified in Eq. (7-1). The results showed when the type-I beams were loaded cyclically up to 2% of rotation angle, the failure mode of beam bar bond slip, without attaining their desired flexural strength, was found. However, the type-II beams exhibited a better seismic performance that meets the requirement of ACI 374.2R-13 (ACI, 2013). That is, the cut-off SD 690-D32 beam bars for type-II beams did not slip until being loaded to 3.5% of rotation angle where the bars were stressed to 1.1f_y. Therefore, from the above tested results, it is recommended the specified concrete strength be larger than 50 MPa in case the high strength SD 690 bars are adopted. Otherwise, the unexpected failure of beam bar slipping might occur when lower concrete strength was used to the new RC beams.

For studies on bond of the high-strength reinforced concrete, the modified factors of development lengths of bars in compression have not been studied yet. Therefore, the modified factors of ACI 318-14 for the calculations of development lengths of bars in compression, as given in Table C7-2, are adopted.

For studies on bond of the high-strength reinforced concrete, the modified factors of development lengths of bundled bars have not been studied yet. Therefore, the modified factors of ACI 318-14 for the calculations of development lengths of bundled bars are able to be referred.

7.2.2 Development of Standard Hooks

(1) Development of standard hooks in tension

The development length for standard hooks bars in tension, ℓ_{dh} shall be calculated in accordance with Eq. (7-4), and ℓ_{dh} shall not less than $8d_b$ and 150 mm.

$$\ell_{dh} = 0.24 \frac{f_y \psi_e}{\sqrt{f_c'}} d_b \quad \text{(mm, MPa)} \tag{7-4}$$

(2) Development of standard hooks in compression

Standard hooks shall not be considered effective in developing bars in compression.

[Commentary]

Standard hooks for the bend diameters and development length ℓ_{dh} of deformed bars in tension shall conform to Fig. C7-3.

Generally, the failure mode of hooked bars is observed from splitting of the *concrete cover in the plane of the hook or crushing at the inside of the hook. To prevent the failures, the tensile stress on the inside of the hook shall be reduced. The development length ℓ_{dh} is measured from the critical section to the outside end of the hook (Fig. C7-3). The use of anchorage length ℓ_{dh} can be reduced the maximum stress f_y of the reinforcement in the critical section by actions of the bond stresses in the ℓ_{dh} range, so that the tensile stress on the inside of the hook can be decreased without failures.*

For studies on bond of the high-strength reinforced concrete, the modified factors of development lengths of standard hooks in tension have not been studied yet. Therefore, the modified factors of ACI 318-14 for the calculations of development lengths of standard hooks in tension, as given in Table C7-3, are adopted.

Because the SD 690 grade of longitudinal reinforcement will be used for against seismic loading, and the high-strength reinforcement is a brittle metal due to the high content of carbon, standard hook or welded connection is not allowed. Therefore, the high-strength reinforcing bars will be manufactured in a screw deformed shape to generate convenient mechanical anchorages with the threaded end-anchorage grout-filled head.

In compression, hooks are ineffective and may not be used as anchorage.

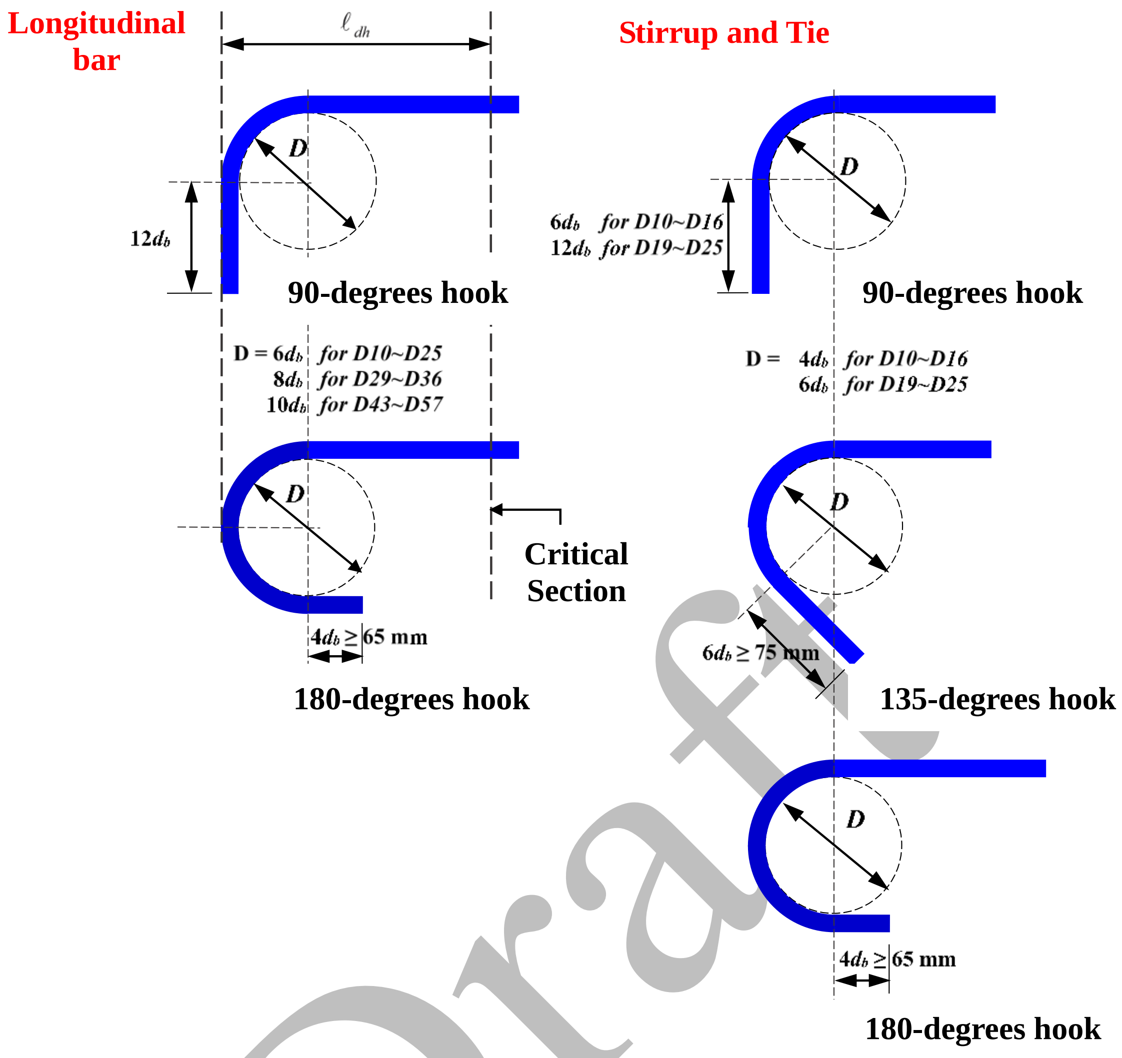

Fig. C7-3 Details of standard hooks.

Table C7-3 Factors used for the development length of standard hook in tension.

Consideration	*Conditions*	*Factors*
Reinforcement coating (ψ_e)	*(1) Epoxy-coated reinforcement* *(2) Uncoated reinforcement*	*1.2* *1.0*
Cover thickness	*For D36 bar and smaller hooks with side cover (normal to plane of hook) not less than 65 mm, and for 90-degree hook with cover on bar extension beyond hook not less than 50 mm.*	*0.7*
Stirrup or tie	*(1) For 90-degree hooks of D36 and smaller bars that are either enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along ℓ_{dh}; or enclosed within ties or stirrups parallel to the bar being developed, spaced not greater than $3d_b$ along the length of the tail extension of the hook plus bend.* *(2) For 180-degree hooks of D36 and smaller bars*	*0.8*

	that are enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than 3d_b along ℓdh. *(3) In (1) and (2), db is the diameter of the hooked bar, and the first tie or stirrup shall enclose the bent portion of the hook, within 2d_b of the outside of the bend.*	
Excess reinforcement	*(1) Development of rebar for f_y is specifically required.* *(2) The factor shall be permitted where reinforcement is in excess of that required by analysis.*	*1.0* $\dfrac{A_s \text{ required}}{A_s \text{ provided}}$

7.2.3 Development of Headed Bars

(1) Development of headed bars in tension

The development length for headed bars in tension, ℓ_{dt} shall be calculated in accordance with Eq. (7-5), and ℓ_{dt} shall not less than 8d_b and 150 mm.

$$\ell_{dt} = 0.192 \frac{f_y \psi_e}{\sqrt{f_c'}} d_b \qquad \text{(mm, MPa)} \tag{7-5}$$

where ψ_e is the coated factor which shall be taken as 1.2 for epoxy-coated reinforcement and 1.0 for other cases.

Use of heads to develop bars in tension shall satisfy (a) through (g):

(a) The specified yield strength of headed bars, f_y, shall not exceed 690 MPa.

(b) Bar size shall not exceed No. 14 (D41).

(c) Net bearing area of head shall not be less than 4A_b.

(d) Clear cover for bar shall not be less than 2d_b.

(e) Clear spacing between bars shall not be less than 2d_b.

(f) Concrete shall be normalweight.

(g) The specified compression strength of concrete shall not exceed 100 MPa.

(2) Development of headed bars in compression.

Heads shall not be considered effective in developing bars in compression.

[Commentary]

The provisions for the development of headed bars in tension shall conform to related requirement of ACI 318-14 code in this chapter. Also, the requirement is verified by some research results (Lin et al., 2010 and 2014). According to the requirements of ACI 318-14 code, the development of headed bars in tension shall meet several conditions. Among these requirements, the four provisions of "bar f_y *shall not exceed 420 MPa", "bar size shall not exceed No. 11 (D36)", "the specified compression strength of concrete shall not exceed 42 MPa", and "clear spacing between bars shall be at least* $4d_b$*" are not practical. It is recommended the provisions have to be adjusted in a view of practical way.*

To study the anchorage performance of headed bars, the series of CCT node tests in beams (Lin et al., 2010 and 2014) were used to investigate the anchorage behavior of headed bars. The test results indicated that the above provisions can be released. These are "the upper limit of specified yield strength f_y *of bars is adjusted to 690 MPa", "the bar size can be used to No. 14 (D41)", "the specified compression strength of concrete for calculating the development lengths is adjusted to 100 MPa", and "clear spacing between bars can be reduced to* $2d_b$*".*

The provisions for developing headed deformed bars give the length of bar, ℓ_{dt}*, measured from the critical section to the bearing face of the head, as shown in Fig. C7-4. For the provision of development lengths of grout-filled headed* *bars in JSCE (2007),* ℓ_{dt} *measured from the front end of the plate nut to the bar end (Fig. C7-4). However, the related research (Lin et al., 2010 and 2014) showed that the expected strengths and anchorage performance of headed bars can be developed by the* ℓ_{dt} *which is measured from the back end of the plate nut to the bar end.*

In compression, heads are ineffective and may not be used as anchorage.

The types of heads for headed bars can be divided into welded, hot-rolled, and grout-filled nut, as shown in Fig. C7-5. According to the requirements in AWS D1.4 (AWS, 2011), the carbon equivalent of high-strength reinforcement which is greater than 0.55% shall not be welded. Additionally, it is recommended that the SD 690 grade which is no upper limit of carbon equivalent in Table C2-3 should use the threaded bars with grout-filled nut for anchoring. The SD 490W and SD 550W steel bars shall conform to the requirements of the welding process even the carbon equivalent is less than 0.55%.

Regardless of types for headed bars, the heads shall comply with the provisions of "Specification for Headed Steel Bars for Concrete Reinforcement" (TCI, 2014c) by Taiwan Concrete Institute.

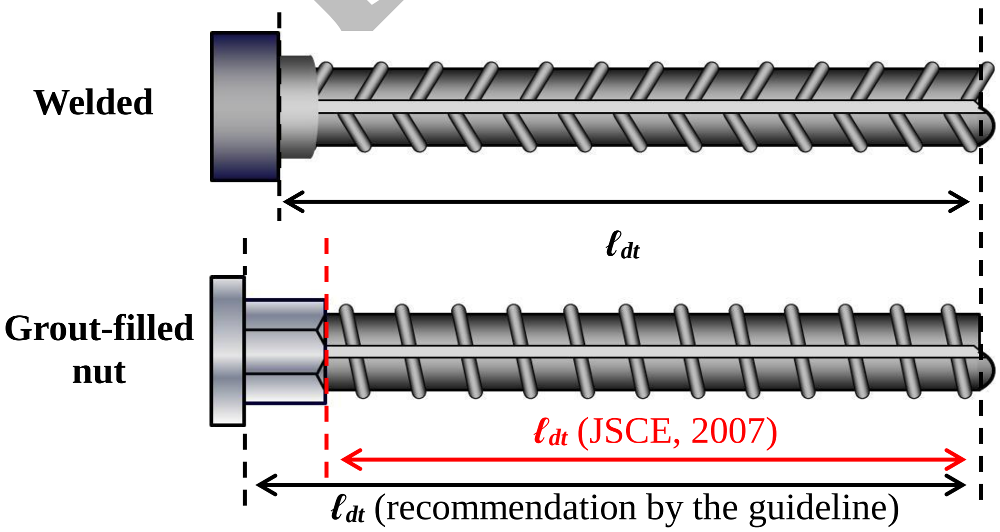

Fig. C7-4 Details of the development length of headed bars.

(a) Welded headed bars

(b) Screw welded headed bars

(c) Hot-rolled headed bars

(d) grout-filled headed bars

Fig. C7-5 The types of anchorage plates for headed bars.

7.3 Splices

Splices of high-strength reinforcement shall satisfy following requirements:

(1) Lap splices shall not be permitted for reinforcing bars with a specified yield strength greater than or equal to 550 MPa.

(2) Weld splices shall not be permitted for reinforcing bars with a specified yield strength greater than or equal to 550 MPa.

(3) Mechanical splices evaluated in compliance with ICC-ES AC133 or equivalent criteria shall be permitted.

[Commentary]

Steel reinforcing bars are typically connected by lap, weld, or mechanical splices. The higher yield strength is used, the larger lap splice length is required. For special seismic systems, lap splices of longitudinal reinforcement are permitted only within regions away from critical sections. The use of lap splices for high-strength reinforcement is impractical because the lap splice length may not be within the central half of the column length. Steel congestion of lap splices may also affect the quality of concrete. Due to insufficient evidence in experiments, lap splices should not be permitted for reinforcement with bar f_y *greater than or equal to 550 MPa.*

It is very difficult to make direct butt weld splices for large-diameter Grade 690 bars in construction sites. The chemical composition of Grade 690 reinforcement usually gives a value of carbon equivalent (C.E.) over 0.55 percent. Heat of welding would reduce the ductility of reinforcement, unless the C.E. of the steel is not to exceed 0.55 percent and welded with compatible welding procedures by licensed welders, as required in AWS D1.4. In practice, butt welding of closely-spaced longitudinal bars in frame members is impractical due to limited space for griping the bars and welding in alignment. Accordingly, butt weld splices are not recommended herein. Friction welding between mechanical couplers and bar ends should be permitted if the welding connections are capable of developing the tensile strength of the bars. The quality of friction welding connections depends on bar chemical compositions, bar end conditions, friction-welding pressure, and cooling time. The alignment of friction-welded couplers also affects the connection strength. Friction-welded couplers for Grade 550 and 690 reinforcing bars may be permitted if experiments could demonstrate that such connections satisfy the requirements of ICC-ES AC 133 (2015) or other equivalent criteria.

Three basic categories of mechanical splices are tension-only, compression-only, and tension-compression mechanical splices. Clearly, tension-compression mechanical splices that can resist both tensile and compressive forces, while the others can only resist tensile or compressive forces. A variety of proprietary couplers or coupling sleeves for mechanical splices are currently available. General descriptions of the relevant features, characteristics, and installation procedures of various available mechanical splices can be found in reports of ACI 439.3R-07 (ACI, 2007) and JSCE (2007). For bar f_y *not exceeding 490 MPa, a variety of mechanical splices are available. To date, a few options of mechanical splices are available for bar* f_y *exceeding 550 MPa.*

For special frame systems, lap splices of longitudinal reinforcement are only permitted within the locations where yielding of the reinforcement is unlikely to occur. This limitation often makes steel congestion in beams and columns. Therefore, mechanical splices are commonly used to make the steel fabrication more practical and cost effective than lap splices. To date, several innovative mechanical couplers and coupling sleeves are developed for the prefabricated steel cages or precast concrete structures (JSCE, 2007).

Mechanical splices have been used in practice for decades. In accordance with ACI 318, Type 1 mechanical splices should develop in tension or compression, as required, a minimum of $\alpha_o f_y$ *of the bar. Type 2 mechanical splices are required to develop the larger of* $\alpha_o f_y$ *and* f_u *for the spliced bars. Type 1 mechanical splices should not be located within a distance equal to twice the member depth from the faces of beam-column joints or from critical sections where yielding of the reinforcement is likely to occur during a seismic event. Type 2 mechanical splices should be permitted at any location. Special inspections and quality assurances are needed to ensure a satisfactory* *performance in the critical locations where Type 2 splices are used. The splice types, locations, and typical installation requirements should be clearly given in design drawings and specifications, or specified by professional engineers.*

Besides the strength requirements, design considerations should include the slip, ductility, locations, clearance, and cover thickness of mechanical splices. In a flexural member, any mechanical splice should not result in a low effective longitudinal stiffness of the spliced bars. In sections where inelastic strains are anticipated, such as in potential plastic hinge sections, the Type 2 mechanical splices should possess adequate ductility to avoid the splice failure. Engineers could verify the performance of mechanical splices by testing mechanical spliced assemblies or structural elements with mechanical splices.

For common practice in Taiwan, mechanical splices are classified as Grade SA or B. The strength, deformability, and ductility of Grade SA mechanical splices should be as good as those of the parent reinforcement. Grade SA mechanical splices should also conform to the strength requirements of Type 2 mechanical splices. Grade B mechanical splices could only conform to the strength requirements of Type 1 mechanical splices.

Lee et al. (2017) evaluated the acceptance criteria and test methods for mechanical splices in Taiwan, and concluded the Public Construction Guideline Chapter 03210-v5.0 (PCC, 2018) may be too stringent for bar f_y *exceeding 490 MPa. For reinforcement Grade 550 and 690, the performance of mechanical splices should be evaluated in accordance with ICC-ES AC133 (2015), ISO 15835 (2009), JSCE (2007), or TCI (2014b). The test methods and requirements in these equivalent criteria are similar but not identical. Ou et al. (2015) tested grouted sleeve splices for Grade 690 reinforcing bars in accordance with TCI (2014b) and AC 133. Cyclic loading tests of precast high-strength reinforced* *concrete columns incorporating the grouted sleeve splices in plastic hinge regions exhibited comparable seismic performance with monolithic counterparts.*

Reference

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[7.2] AWS D1.4, 2011, "*Structural Welding Code - Reinforcing Steel*," American Welding Society, American National Standards Institute.

[7.3] Construction and Planning Agency Ministry of the Interior (CPAMI), 2017, "*Design Specifications for Concrete Structures*," Taiwan. (in Chinese)

[7.4] Construction and Planning Agency Ministry of the Interior (CPAMI), 2017, "*Construction Specifications of RC Structures*," Taiwan. (in Chinese)

[7.5] High-Strength Reinforced Concrete Committee Report, 2014a, "High-Strength Steel Bars for Concrete Reinforcement (SD 550/685/785)," Taiwan Concrete Institute (TCI), Taiwan. (in Chinese)

[7.6] High-Strength Reinforced Concrete Committee Report, 2014b, "Guidelines for Performance Evaluation of Mechanical Splices for High-Strength Steel Reinforcing Bars," Taiwan Concrete Institute (TCI), Taiwan. (in Chinese)

[7.7] Hung-Jen Lee, Cheng-Cheng Chen, Chien-Chung Chen, and Chia-Jung Chang, 2017, "Experimental Investigations on the Acceptance Criteria of Mechanical Splices for Reinforcing Bars in Building Structures," Report No. 106301070000G0032, Architecture and Building Research Institute, Ministry of the Interior, Taipei, Taiwan, 158 pp. (in Chinese)

[7.8] ICC-ES AC133, 2015, "*Acceptance Criteria for Mechanical Connector Systems for Steel Reinforcing Bars (AC133)*," ICC Evaluation Service, Whittier, California, 9 pp.

[7.9] ISO 15835, 2009, "*Steels for the Reinforcement of Concrete — Reinforcement Couplers for Mechanical Splices of bars — Part 1: Requirements, Part 2: Test methods*," ISO copyright office, Switzerland, 36 pp.

[7.10] JSCE, 2007, "*Recommendations for Design, Fabrication and Evaluation of Anchorages and Joints in Reinforcing Bars*," Japan Society of Civil Engineering, Tokyo, 561 pp. (in Japanese)

[7.11] Ou, Yu-Chen, Harun Alrasyid, Zachary B. Haber, and Hung-Jen Lee, 2015, "Cyclic Behavior of Precast High-Strength Reinforced Concrete Columns." *ACI Structural Journal*, Vol. 112, No. 6, pp. 839-850.

Chapter 8 Comparison of The Provisions Differences

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Based on the previous chapters of this design guideline, this chapter summarizes and compares the differences between this guideline and "Design Specifications for Concrete Structures." (CPAMI, 2017)

Design Specifications for Concrete Structures (CPAMI, 2017) (SI units)	**This Design Guideline (SI units)**
(1) The coefficient for equivalent rectangular stress block of concrete	
$\alpha_1 = 0.85$	$0.7 \le \alpha_1 = 0.85 - 0.0022\left(f_c^{'} - 55\right) \le 0.85$
(2) Nominal shear strength from transverse reinforcement of the beam	
$V_s = \dfrac{A_v f_{yt} d}{s} \le 0.66\sqrt{f_c^{'}} b_w d$ $f_{yt} \le 420\,\text{MPa}$	$V_s = \dfrac{A_v f_{yt} d}{s} \le 0.66\sqrt{f_c^{'}} b_w d$ $f_{yt} \le 600\,\text{MPa}$
(3) Requirements for spacing of transverse reinforcement of the beam	
(A) The first hoop should be located not more than 50 mm from the face of a supporting column. Spacing of the hoops should not exceed the least of (a) through (d).: (a) *d*/4. (b) eight times the diameter of the smallest primary flexural reinforcing bars. (c) Twenty-four times the diameter of the smallest hoop. (d) 300 mm. (B) Where hoops are not required, stirrups with seismic hooks at both ends should be spaced at a distance not more than *d*/2 throughout the length of the beam.	(A) The first hoop should be located not more than 50 mm from the face of a supporting column. Spacing of the hoops should not exceed the least of (a) through (c).: (a) *d*/4. (b) Five times the diameter of the smallest primary flexural reinforcing bars. (c) 150 mm. (B) Where hoops are not required, stirrups with seismic hooks at both ends should be spaced at a distance not more than the lesser of *d*/2 and 300 mm. (C) Spacing of transverse reinforcement enclosing the lap-spliced bars should not exceed the lesser of *d*/4 and 100 mm.

Design Specifications for Concrete Structures (CPAMI, 2017) **(SI units)**	**This Design Guideline** **(SI units)**
(4) The axial strength at zero eccentricity *P_0* of the column	
$P_0 = 0.85 f_c'(A_g - A_{st}) + f_y A_{st}$ $f_y \le 420\,\text{MPa}$	$P_0 = \alpha_1 f_c'(A_g - A_{st}) + f_y A_{st}$ $f_y \le 600\,\text{MPa}$
(5) Limits of material strengths for calculating shear strength of the column	
(A) The value of used in the calculation of shear strength provided by concrete should not be larger than $\sqrt{70\ \text{MPa}}$. (B) The steel stress used in the calculation of shear strength provided by shear reinforcement should not be larger than 420 MPa.	(A) The value of used in the calculation of shear strength provided by concrete should not be larger than $\sqrt{100\ \text{MPa}}$. (B) The steel stress used in the calculation of shear strength provided by shear reinforcement should not be larger than 600 MPa.
(6) Detailed equations for the nominal shear strength provided of the column by concrete	
$V_c = \left(0.16\sqrt{f_c'} + 17\rho_w \dfrac{V_u d}{M_m}\right) b_w d$ $\le V_c = 0.29\sqrt{f_c'}\, b_w d$ $M_m = M_u - N_u\left(\dfrac{4h-d}{8}\right)$	$V_c = \left(0.16\sqrt{f_c'} + 17\rho_w \dfrac{V_u d}{M_m}\right) b_w d$ $\le V_c = 0.29\alpha\sqrt{f_c'}\, b_w d \sqrt{1 + \dfrac{2N_u}{\alpha\sqrt{f_c'}\, b_w d}}$ $M_m = M_u - N_u\left(\dfrac{4h-d}{8}\right)$ $\alpha = \left(1 - 0.85\sqrt{\dfrac{N_u}{A_g f_c'}}\right)$ ， $0 \le \dfrac{N_u}{A_g f_c'} \le 0.6$
(7) The minimum amount of transverse reinforcement *$A_{v,min}$* for the shear strength	
$A_{v,min}$ shall be the greater of (a) and (b): (a) $A_{v,\min} = 0.062\sqrt{f_c'}\dfrac{b_w s}{f_{yt}}$ (b) $3.5\dfrac{b_w s}{f_{yt}}$ $f_{yt} \le 420\,\text{MPa}$	$A_{v,\min} = \dfrac{0.38 V_c s}{f_{yt} d}\beta$ $1.0 \le \beta = \dfrac{3N_u}{A_g f_c'} + 0.4 \le 1.3$ $f_{yt} \le 600\,\text{MPa}$

Design Specifications for Concrete Structures (CPAMI, 2017) **(SI units)**	**This Design Guideline** **(SI units)**
(8) Amount of transverse reinforcement for confinement of the column	
the amount of rectilinear hoop reinforcement shall not be less than required by (a) and (b): (a) $\frac{A_{sh}}{sb_c} = 0.3\frac{f_c'}{f_{yt}}(\frac{A_g}{A_{ch}} - 1)$ (b) $\frac{A_{sh}}{sb_c} = 0.09\frac{f_c'}{f_{yt}}$ $f_{yt} \leq 420\,\text{MPa}$	If the axial load applied on the column is less than or equal to $0.3A_g f_c'$ and the specified compressive strength of concrete is less than 70 MPa, the amount of rectilinear hoop reinforcement shall not be less than required by (a) and (b): (a) $\frac{A_{sh}}{sb_c} = 0.3\frac{f_c'}{f_{yt}}(\frac{A_g}{A_{ch}} - 1)$ (b) $\frac{A_{sh}}{sb_c} = 0.09\frac{f_c'}{f_{yt}}$ If the axial load applied on the column is greater than or the specified compressive strength of concrete is greater than 70 MPa, the amount of rectilinear hoop reinforcement shall not be less than required by (a), (b) and (c): (c) $\frac{A_{sh}}{sb_c} = 0.2k_f k_n \frac{P_u}{f_{yt}A_{ch}}$ where $k_f = \frac{f_c'}{175} + 0.6 \geq 1.0$ $k_n = \frac{n_l}{n_l - 2}$ $f_{yt} \leq 800\,\text{MPa}$
(9) Requirements for spacing of transverse reinforcement of the column	
Spacing of transverse reinforcement shall not exceed the smallest of (a) through (c): (a) One-fourth of the minimum column dimension. (b) Six times the diameter of the smallest longitudinal bar. (c) $s_0 = 100 + \left(\frac{350 - h_x}{3}\right)$ 。 where s_0 shall not exceed 150 mm and need not be taken less than 100 mm.	Spacing of transverse reinforcement shall not exceed the smallest of (a) through (c): (d) One-fourth of the minimum column dimension. (e) Five times the diameter of the smallest longitudinal bar. (f) $s_0 = 100 + \left(\frac{350 - h_x}{3}\right)$ 。 where s_0 shall not exceed 150 mm and need not be taken less than 100 mm.

Design Specifications for Concrete Structures (CPAMI, 2017) (SI units)	**This Design Guideline (SI units)**
(10) The stress $\alpha_o f_y$ in the flexural tensile reinforcement of the beams framing into the joint	
The stress $\alpha_o f_y$ in the flexural tensile reinforcement of the beams framing into the joint shall be assumed $1.25 f_y$.	The stress $\alpha_o f_y$ in the flexural tensile reinforcement of the beams framing into the joint shall be in accordance with: $\alpha_o = 1.25$ --- SD 490W, SD 550W $\alpha_o = 1.20$ --- SD 690
(11) Nominal joint shear strength	
$V_n = 0.083\gamma\sqrt{f_c^{'}}A_j$ γ =1.7 for joints confined by beams on all four faces. γ =1.2 for joints confined by beams on three faces or on two opposite faces. γ =1.0 for other cases. Effective joint width (b_j) shall not exceed the lesser of (a) and (b): (a) $b+h$ (b) $b+2x$	$V_n = 0.083\gamma\sqrt{f_c^{'}}A_j$ The constant γ is given in Table 5-2. Effective joint width (b_j): $b_j = b + x_1 + x_2 \le b_{col}$ where x_1 and x_2 are the effective width beyond the beam web to the column side on each side of the beam web. The width of x_1 or x_2 shall not be taken greater than $h_c/4$.
(12) Minimum column dimension parallel to the beam reinforcement extends through a beam-column joint	
Where longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall be at least 20 times the diameter of the largest longitudinal beam bar.	Where longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall be at least the larger of (a) and (b). (a) $\dfrac{\alpha_o f_y}{4\sqrt{f_c^{'}}}$ (b) $20d_b$

Design Specifications for Concrete Structures (CPAMI, 2017) **(SI units)**	**This Design Guideline** **(SI units)**
(13)Development Length Requirements for Beam Bars Terminated Within a Joint	
(A) 90º standard hooks: The development length ℓ_{dh} of a beam longitudinal bar terminating in a standard hook within a joint shall be calculated by this equation, and ℓ_{dh} shall be at least the greatest of $8d_b$, 150 mm. $\ell_{dh} = \dfrac{f_y d_b}{5.4\sqrt{f_c^{'}}}$ $f_{yt} \leq 420\,\text{MPa}$ $f_c^{'} \leq 70\,\text{MPa}$	(A) 90º standard hooks: The development length ℓ_{dh} of a beam longitudinal bar terminating in a standard hook within a joint shall be calculated by this equation, and ℓ_{dh} shall be at least the greatest of $8d_b$, 150 mm, and 3/4 of the column depth. $\ell_{dh} = \dfrac{f_y d_b}{5.4\sqrt{f_c^{'}}}$ (A) Headed bars： headed deformed bars satisfying following conditions (a) through (c), the development length ℓ_{dt} of a headed bar within a joint shall be calculated by this equation, and ℓ_{dt} shall be at least the greatest of $8d_b$, 150 mm, and 3/4 of the column depth. $\ell_{dh} = \dfrac{f_y d_b}{5.2\sqrt{f_c^{'}}}$ (a) Net bearing area of head A_{brg} shall be at least $4A_b$. (b) Value of f_c' used to calculate development length shall not exceed 100 MPa. (c) Clear spacing between bars shall be at least $2d_b$. $f_{yt} \leq 690\,\text{MPa}$ $f_c^{'} \leq 100\,\text{MPa}$

Design Specifications for Concrete Structures (CPAMI, 2017) **(SI units)**	**This Design Guideline** **(SI units)**
(14)The nominal shear strength of the wall	
$V_n = A_{cv}(\alpha_c\sqrt{f_c^{'}} + \rho_t f_{yt}) \leq 0.67 A_{cv}\sqrt{f_c^{'}}$ $f_{yt} \leq 420\,\text{MPa}$ $f_c^{'} \leq 70\,\text{MPa}$	$V_n = A_{cv}(\alpha_c\sqrt{f_c^{'}} + \rho_t f_{yt}) \leq 0.67 A_{cv}\sqrt{f_c^{'}}$ $f_{yt} \leq 600\,\text{MPa}$ $f_c^{'} \leq 100\,\text{MPa}$
(15)Straight development of reinforcement in tension, *ℓ* ℓ_d	
$\ell_d = 0.9\dfrac{f_y}{\sqrt{f_c^{'}}}\dfrac{\psi_t\psi_e\psi_s}{\left(\dfrac{c_b+K_{tr}}{d_b}\right)}d_b \geq 300\,mm$ $1.0 \leq \left(\dfrac{c_b+K_{tr}}{d_b}\right) \leq 2.5$ $K_{tr} = \dfrac{A_{tr} f_{yt}}{105 sn}$ $f_{yt} \leq 420\,\text{MPa}$ $f_c^{'} \leq 70\,\text{MPa}$	$\ell_d = 0.9\dfrac{f_y}{\sqrt{f_c^{'}}}\dfrac{\psi_t\psi_e\psi_s}{\left(\dfrac{c_b+K_{tr}}{d_b}\right)}d_b \geq 300\,mm$ $1.0 \leq \left(\dfrac{c_b+K_{tr}}{d_b}\right) \leq 2.5$ $K_{tr} = \dfrac{40A_{tr}}{sn}$ the bond strengths of deformed bars and threaded bars with R_r values that exceed 0.12 and 0.17 are all good enough to develop the required strengths when the strengths of concrete are not limited to 70 MPa, even the compressive strength of concrete is increased up to 100 MPa. $f_{yt} \leq 690\,\text{MPa}$ $f_c^{'} \leq 100\,\text{MPa}$
(16)Development of headed bars, ℓ_{dt}	
No provisions.	$\ell_d = 0.192\dfrac{\psi_e f_y}{\sqrt{f_c^{'}}}d_b \geq 8d_b, 150\,mm$ $f_{yt} \leq 690\,\text{MPa}$ $f_c^{'} \leq 100\,\text{MPa}$